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THE RELATIONSHIP BETWEEN ENERGY AND STRENGTH FOR A REMOULDED, MODIFIED, SILTY SOIL

by

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A THESIS

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FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "THE RELATIONSHIP BETWEEN ENERGY AND STRENGTH FOR A REMOULDED, MODIFIED, SILTY SOIL", submitted by LAWRENCE ALEXANDER BALANKO in partial fulfilment of the requirements of the degree of Master of Science.

Date: October, 22, 1968

ABSTRACT

The purpose of this thesis was to provide additional information concerning any relationships that may exist between soil strength characteristics and energy as applied in a consolidation process. The consolidated-undrained triaxial test with pore pressure measurements was utilized with a maximum confining pressure of 8 Kg/sq cm. The basic soil was a silt containing no clay minerals. The influence of plasticity on strength and energy absorption was assessed by kaolin and montmorillonite modifications of the basic soil. Tests were performed on normally consolidated and over-consolidated samples to study the influence of stress history.

For the soils and test conditions employed, it was shown that a linear proportionality existed between strength and applied energy for normally consolidated specimens. For over-consolidated specimens this relationship was found to be non-linear and appeared to become increasingly so as the plasticity of the soil increased. It was concluded that energy absorption and retention characteristics are a function of the mineralogy of the soil. An inter-relationship existed between plasticity, the effective angle of shearing resistance and energy stored and applied. It was found that with increasing plasticity the effective angle of shearing resistance decreased and the energy increased.

It is recommended that future investigations employ higher confining pressures and longer loading intervals. Confining pressures in the order of 1500 to 2000 pounds per square inch would more closely simulate field conditions. Tests should ultimately progress from the use of laboratory manufactured samples to the use of undisturbed field specimens.

ACKNOWLEDGEMENTS

The author extends sincere thanks to the National Research Council of Canada who provided the financial assistance necessary to make this program possible.

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To the numerous others who gave advice and provided assistance, many thanks.

TABLE OF CONTENTS

TABLE OF CONTENTS CONT.

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES CONT.

CHAPTER I

INTRODUCTION

The basic concept of this research arose from the work of Bjerrum (1967)*. In his presentation of the Third Terzaghi Lecture he presented a logical hypothesis, which quantitatively explained the behaviour of over-consolidated clays and the mechanism of long term slope failures in these clays. His hypothesis of progressive failure in slopes is based on the progressive decrease in strength due to the breakdown of diagenetic bonds and subsequent release of locked in strain energy.

In support of this hypothesis, Brooker (1967) presented quantitative evidence gathered from large scale consolidation tests on five remoulded soils. From the results of his tests he concluded that a relationship exists between the coefficient of earth pressure at rest, K_o , the over-consolidation ratio, OCR , and strain energy. He also pointed out that the influence of weathering on over-consolidated soils, as inferred from slaking tests, appeared to be a function of strain energy as well as mineralogy. The strain energy considered by Brooker is stored strain energy and is computed as the area bounded by the stress-strain curve obtained from anisotropic consolidation tests.

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* References listed alphabetically in "List of References".

Of the diagenetic processes, compression appears to be the dominant factor in the early stages of diagenesis. Locker (1968)* has shown that the soft rocks or shales that exhibit the greatest degree of disintegration on weathering, as inferred from laboratory freeze-thaw and wet-dry tests, are those that he terms "compaction" shales. These shales are thought to derive their strength characteristics almost entirely from the compression process, and from the point of view of soil mechanics appear to present the most serious problems. Locker's findings coupled with those of Brooker and Bjerrum suggest that there is a relationship between the compression process and strength, and strain energy.

In view of the dominance of the compression process in the early phases of diagenesis, the research of this thesis was undertaken in anticipation that additional information would be provided concerning relationships that may exist between strength characteristics of soils and energy applied, stored, or liberated in a consolidation process. Not only does compression appear to be prominent in diagenesis, it also is the only factor that may be readily studied as a separate entity in the laboratory on remoulded samples.

In this study, samples were consolidated isotropically in the triaxial cell and tested in undrained compression with pore pressure measurements. The soil chosen for this program was a lacustrine silt taken from an exposed cut on the Alaska Highway, and is essentially free of clay minerals. In order to study the effects

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* Personal Communication, concurrent research, Ph.D. program, University of Alberta.

of clay minerals on strength and energy absorption characteristics, two modifications of the original soil were made by the addition of 10 percent by weight kaolin or montmorillonite to portions of the original soil.

For each soil type a set of normally consolidated and a set of over-consolidated samples were prepared and tested. In this way plots could be made of final void ratio versus consolidation pressure, and the energy associated with volumetric deformation could be computed, and compared to the strength obtained in triaxial compression.

The following chapters present a brief review of pertinent literature, sample preparation, and an examination and discussion of the test technique. The results of the testing program are presented and discussed in light of the factors that affect the results. Conclusions are then drawn in light of the discussion and are followed by recommendations for further research.

CHAPTER II

LITERATURE REVIEW

2.1 Strain Energy in Ideal Elastic Materials

A material body, such as steel, consists of molecules between which forces are acting (Timoshenko, 1955). These forces resist the change in shape of a body which external forces tend to produce. Under the application of external forces, the particles of the body are displaced until equilibrium is established between the external and internal forces. During the deformation the external forces acting upon the body do work which may be completely or partially transformed into potential energy of strain.

In the case of elongation of a prismatic bar, and if the strain remains within the elastic limit, the work done will be completely transformed into potential energy which can be recovered during a gradual unloading of the strained bar. If the final magnitude of the load is P and the corresponding elongation is δ , the tensile test diagram of FIGURE II.1 may be derived (Timoshenko, 1955).

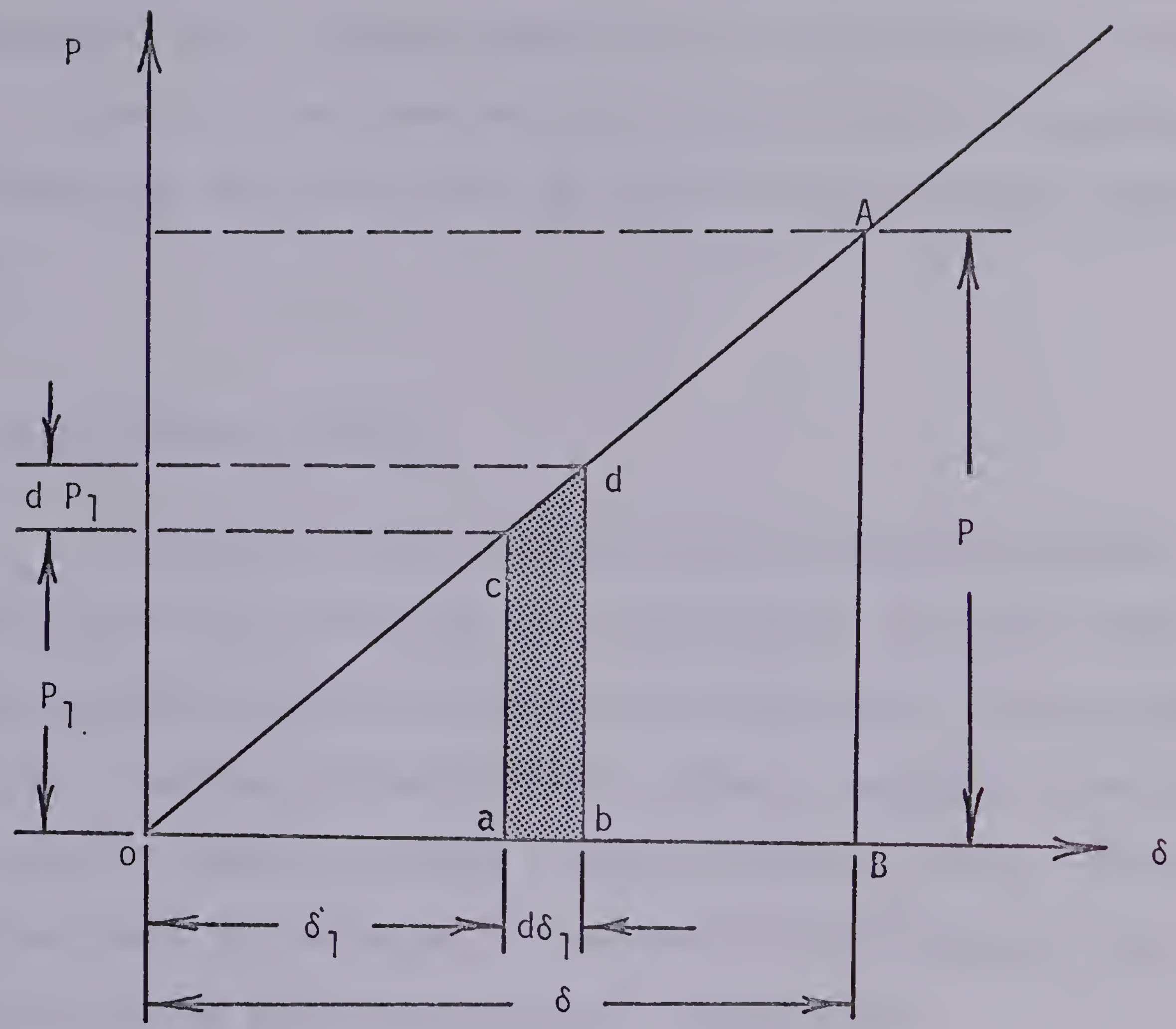

FIGURE II.1 TENSILE TEST DIAGRAM [AFTER TIMOSHENKO, 1955]

The symbol P_1 represents an intermediate value of the load and δ_1 the corresponding elongation. An increment of dP_1 causes a corresponding incremental elongation $d\delta_1$. If allowance is made for the increase of P_1 , the work done during the elongation is represented by the area abcd . The total work done in loading from 0 to P is the summation of all such elemental areas and is given by the area under the stress-strain curve, that is OAB . This represents the total energy stored in the bar (Timoshenko, 1955).

Since most common metals exhibit elastic behaviour over at least a portion of their stress range, equations can be easily

formulated which relate stress, strain, and energy over this range (Timoshenko, 1955). However, most soils tend to exhibit very little elastic behaviour, and therefore, much more difficulty is encountered in formulating simple equations that describe strain energy relationships.

2.2 Strain Energy in Soils

Although the strain energy concept as applied to elastic materials has been established for a considerable time, very little progress has been made with regard to the application of this concept to soils. However, Bjerrum (1967) has recently prompted interest in this regard. Bjerrum presented a logical hypothesis which quantitatively explains the behaviour of over-consolidated clays and the mechanism of long term slope failures in these soils.

It is not within the scope of this thesis to present Bjerrum's hypothesis in detail. However, his hypothesis of progressive slope failure is based on the premise that a decrease in strength occurs due to the progressive break-down of diagenetic bonds with the release of locked-in strain energy.

In the consolidation process, a portion of the energy supplied to the soil-water-air system is recoverable and the remainder is irrecoverable. The energy that is irrecoverable is that lost primarily due to friction, particle displacement, and particle fracture or particle strain beyond the elastic limit. The recoverable portion of the energy supplied is thought to be primarily the result of deformation of the clay particles. Bjerrum (1967, pp 11) states:

> "The amount of recoverable strain energy depends on the consolidation pressure and the properties of the clay and, in general, the more plastic the clay the greater the recoverable strain energy."

The formation of diagenetic bonds results in a restraint of a portion of the recoverable strain energy. On destruction of the bonding, this portion of the energy will be recovered. However, it is not known if the process is completely reversible, as some of the energy may be expended in the diagenetic processes responsible for the bonding.

Of the four main diagenetic processes, compression appears to be the dominant factor in the early stages of diagenesis. In the process of lithification, compression appears to be the only factor that lends itself to laboratory investigation. Other factors of diagenesis such as cementation and recrystallization involve a time factor of such magnitude that it precludes their study using remoulded materials and laboratory consolidation techniques.

Brooker (1967) gathered quantitative evidence from a series of high pressure, anisotropic consolidation tests on remoulded samples which supports the strain energy hypothesis presented by Bjerrum. FIGURE II.2 presents the stress-strain curves obtained by Brooker for the five soils tested. It is evident that the stress-strain curves presented in FIGURE II.2 are not linear, and that it would be extremely difficult to formulate equations representing the strain energy under the curves, as in the case of ideal elastic materials. However, as Brooker points out the area bounded by the loading and unloading branches of the curves represents the strain energy per unit volume absorbed during loading and unloading.

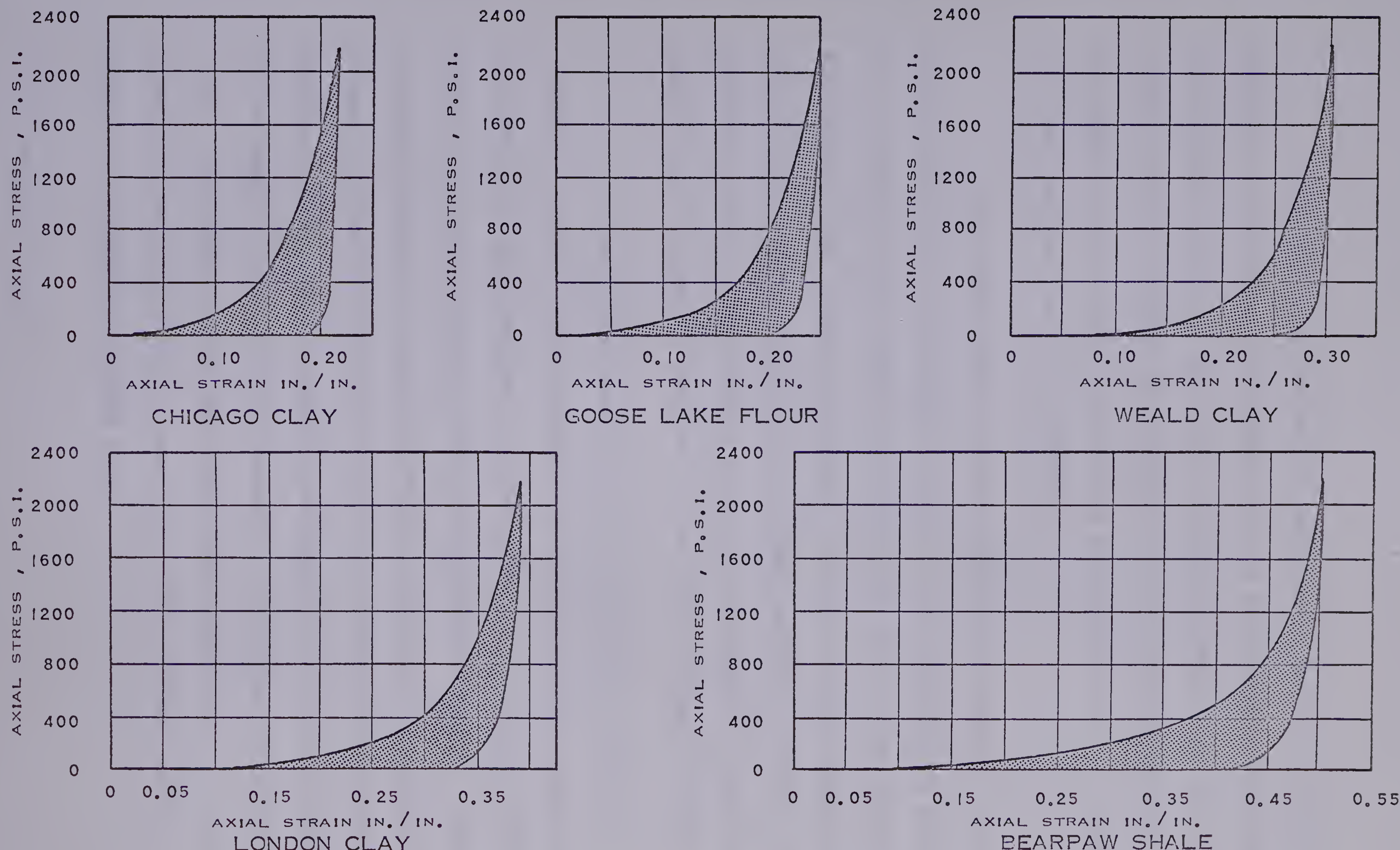

FIGURE II.2 STRESS-STRAIN RELATIONSHIPS FOR ONE-DIMENSIONAL CONSOLIDATION TESTS

[AFTER BROOKER, 1967]

An analysis of the results obtained in terms of stored strain energy has led Brooker to conclude that the most definite relationships exist between values of K_o, OCR, and strain energy. He also concludes that the influence of weathering on over-consolidated soils as suggested by the results of slaking tests appears to be a function of strain energy as well as mineralogy.

It has been shown by Locker (1968)* that the soft rocks or shales that exhibit the greatest degree of disintegration on weathering, as inferred from freeze-thaw and wet-dry tests in the laboratory, are the so-called "compaction" shales. These shales are thought to derive their strength characteristics almost entirely from the compression process. Because these shales lose strength so readily on weathering, they present problems in the field of slope stability.

In view of the dominance of the compression process in diagenesis and the results presented by Bjerrum, Brooker, and Locker, this research was undertaken in anticipation that additional information would be provided concerning energy-strength relationships. The present study employed a somewhat different approach to that taken by Brooker, in that isotropic consolidation was employed as the compression process. In this way a volumetric deformation or strain is imparted to the sample by the confining pressure. If the final void ratio, e_f, is plotted against the consolidation pressure, σ_3, a curve similar to the stress-strain curves obtained by Brooker results. This curve is illustrated in FIGURE II.3.

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* See footnote page 2.

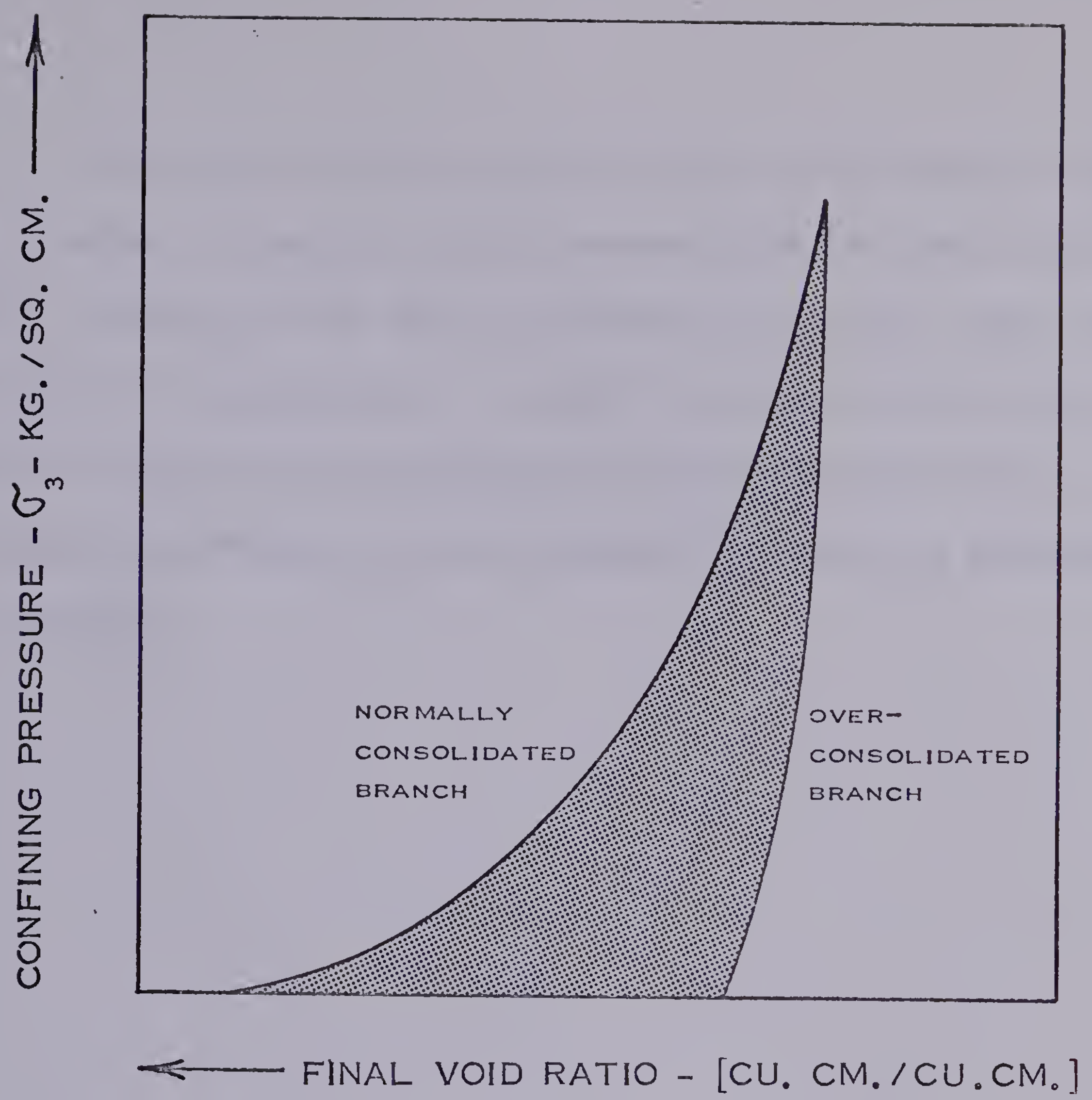

FIGURE II.3 TYPICAL STRESS-VOID RATIO RELATIONSHIP FOR ISOTROPICALLY CONSOLIDATED SOILS

The area bounded by the normally consolidated and over-consolidated branches of the curve has units of Kg-cm/cu cm , which is units of strain energy per unit volume. Although FIGURE II.3 is similar to the curves presented by Brooker, it should be pointed out that the strain energy computed here is a volumetric strain energy associated with isotropic consolidation, as opposed to that associated with anisotropic consolidation. In order to avoid confusion in this regard, the author will simply refer to the energy computed from this type of curve as either energy applied, energy stored, or energy liberated.

In order to obtain a curve similar to that shown in FIGURE II.3, a number of samples must be prepared with the same initial properties. However, since this is difficult to achieve, some scatter in the results can be expected. In order to cope with this scatter, the method of least squares was employed to obtain best fit curves. Details concerning the presentation and treatment of results is presented in later chapters.

CHAPTER III

SAMPLE PREPARATION AND THE TRIAXIAL TEST

3.1 Soil Used in Program

The soil used in this program was obtained from an exposed highway cut at approximately Mile 980 of the Alaska Highway. Approximately 30 pounds of this soil in an air-dried condition was made available to the author. The soil when received was in a very lumpy condition and was passed through two mechanical crushers, and subsequently sieved through a U.S. Number 40 standard sieve prior to using.

Classification tests, performed in accordance with ASTM procedures, yielded the results shown in TABLE III.1. The results of an x-ray diffraction analysis performed on a sample of the soil by the Alberta Research Council are also included in the table.

TABLE III.1

SUMMARY OF CLASSIFICATION TESTS ON ORIGINAL SOIL

Test	Result
Specific Gravity	2.67
Atterberg Limits	
Liquid Limit	21.9 %
Plastic Limit	19.0 %
Plasticity Index	2.9 %
Grain Size Distribution[1]	
% Sand Sizes	7-8
% Silt Sizes	91-89
% Clay Sizes	2-3
Mineralogical Composition[2]	
Quartz	30 %
Plagioclase Feldspar Biotite Calcite Amphibole	Approximately evenly Distributed
Chlorite	Trace

1. M.I.T. Grain Size Scale

2. As determined by the Alberta Research Council on material finer than 0.42 mm

Since no clay minerals were identified in the analysis of the entire sample, no attempt was made to isolate the clay size fraction and perform an x-ray diffraction analysis on it. For all practical purposes the soil can be considered free of clay minerals and has been classified as an inorganic silt of low compressibility (ML) according to the Unified Classification system. This soil exhibits a very high degree of dilatancy.

3.2 Soil Modifications

In order to investigate the effect of clay minerals the composition of the original soil was changed by the addition of a commercially prepared kaolin and a commercially prepared montmorillonite. The kaolin used was acid washed by the manufacturer and therefore is essentially the pure clay mineral kaolinite. The montmorillonite used is a drilling mud available under the trade name "Magcogel", and is almost entirely montmorillonite with some barite as an additive.

As mentioned previously the air-dried silt was mechanically broken down to pass a No. 40 sieve. In order to ensure the availability of an adequate supply of modified soil the silt which had been used in the tests and subsequently oven-dried was also mechanically broken down, resieved through the No. 40 sieve, and recombined with the air-dried soil. Ten percent by weight kaolin or montmorillonite were then added to two equal proportions of this silt. In order to ensure complete dispersal of the kaolin and montmorillonite each was also sieved through a No. 40 sieve, combined with the silt in an air-

dried condition, and mixed with a mechanical mixer until dispersal, as ascertained by visual inspection, was complete. As an added measure to ensuring a uniform soil mixture, a sufficient quantity of distilled water was added to each modification to yield a very thin slurry. The slurry was then mixed in the mechanical mixer for a period of approximately three hours. After mixing, each slurry was placed in a polyethylene bucket and allowed to air-dry until the moisture content was lowered to a level suitable for forming the triaxial specimens.

In order to ascertain the properties of the modifications, routine classification tests were performed on each of the soil modifications. The results of these tests are summarized in TABLE III.2. Grain size distribution and x-ray diffraction analyses were not considered necessary for the modifications, as both modifying agents exist entirely in the form of clay size particles and their mineralogy has been established with a reasonable degree of certainty.

TABLE III.2

SUMMARY OF CLASSIFICATION TESTS ON SOIL MODIFICATIONS

Kaolin modified silt (10% by weight)	Specific gravity	2.75
	Atterberg Limits	
	Liquid Limit	23.0 %
	Plastic Limit	16.4 %
	Plasticity Index	6.6 %
Montmorillonite Modified silt (10% by weight)	Specific gravity	2.73
	Atterberg Limits	
	Liquid Limit	45.8 %
	Plastic Limit	22.6 %
	Plasticity Index	23.2 %

From the results in TABLE III.2 it is apparent that the kaolin modified silt may be classified as an inorganic silt of low plasticity and the montmorillonite modified silt as a clay of medium plasticity. As an added aid in visualizing the plasticity characteristics and comparing the index properties of the modifications with those of the original soil the results of the Atterberg limit tests have been plotted in FIGURE III.1.

3.3 Moulding Triaxial Specimens

Triaxial specimens were moulded using a one-dimensional consolidation apparatus consisting of inner lucite tubes, 34 millimetres inside diameter and 134 millimetres long. A porous stone covered by two pieces of filter paper was wedged in the bottom of the tubes in order to retain the soil samples and to provide adequate end drainage. Vertical drainage aids could not be used, and in fact were not required except on samples of the montmorillonite modification.

Lucite loading caps with drainage aids and filter paper were used to transfer load to the soil within the tube. Each assemblage was placed in 70 millimetre diameter lucite containers with annular spaces between cylinder and container.

Static loads were applied by means of lever arms from which weights could be suspended. In order to ensure a greater degree of similarity between specimens, six samples were prepared simultaneously.

Sample preparation consisted of the placement of the soil slurry in the lucite consolidation tubes by means of a spatula, and

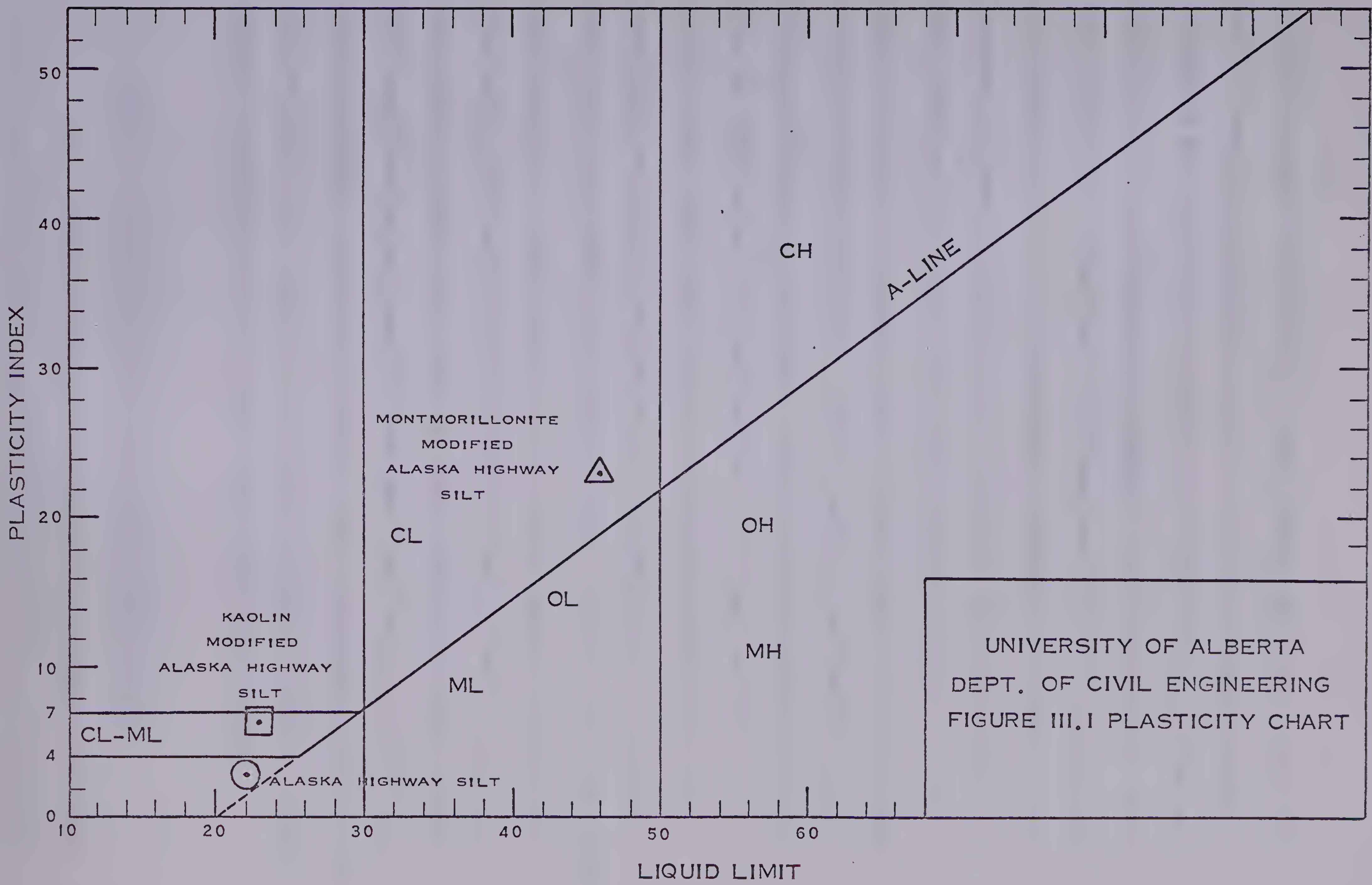

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FIGURE III.1 PLASTICITY CHART

tapping the tubes plus soil on a hard surface after each addition of soil to remove entrapped air. It was found that placement and de-airing of the soil could be greatly facilitated if the slurries were placed at moisture contents approximately 10 to 15 percent in excess of the liquid limit. When the lucite tubes were filled to approximately 1/2 inch of the top the load head was positioned, the annular volume between cylinder and container was filled with distilled water, and the lever arm was brought into contact with the load head. A dial gauge was adjusted to measure vertical displacement of the load head. Plots of observed deflections versus log-time permitted the progress of consolidation to be followed, and hence the determination of the time of 100% theoretical consolidation. Four load increments of 0.188, 0.375, 0.750 and 1.50 Kg/sq cm were used, with full primary consolidation being allowed under each increment for all specimens other than the montmorillonite modifications. In the case of the latter, the time rate of consolidation was too slow to permit 100% consolidation under each load increment within the time available, and only the final load increment was allowed to reach 100% consolidation. After 100% consolidation had been achieved under the final load increment, the samples were reversed and the final load increment was applied from the other end. This step was carried out in order to ensure a greater degree of homogeneity of the samples over their entire length.

3.4 Mounting and Preparation of the Triaxial Test Specimen

The procedure followed in mounting the specimen was similar to that described in the Norwegian Geotechnical Institute Publication

No. 21 (Andresen et al, 1957). However, since the trimming process differs markedly from that given in Publication No. 21, and some aspects of the mounting procedure are also different, a detailed description of the procedure followed is given in APPENDIX A, and only a brief summary is given here. The specimen was trimmed to approximately 34.0 mm diameter by 80.0 mm in length. A saturated, slotted filter paper was wrapped around the specimen and the specimen was placed on a filter paper and porous stone on the pedestal of the triaxial cell. After placing the loading cap in position the entire assembly of load cap, specimen, and pedestal was encased by a rubber membrane and secured at each end with three "O" rings. The exterior of the membrane was coated with a layer of silicone grease to prevent air entry into the sample. The triaxial cell was filled with distilled water and topped with an oil seal approximately one-half inch thick. Consolidation of all specimens was allowed under a back pressure of 1 Kg/sq cm with volume change readings being taken with a twin-burette volume change indicator. A single load increment was used in all cases. When theoretical 100 percent consolidation had been reached the volume change indicator valve was shut off and the triaxial cell transferred to the loading press.

3.5 The Triaxial Test

A detailed description of the equipment and procedure used to perform an undrained triaxial test is given in the Norwegian Geotechnical Institute Publication (Andresen et al, 1957; NGI No. 21, 1957), therefore, the following description is brief.

In contrast to the back pressuring procedure outlined in the NGI Publication, the procedure used by Bishop and Henkel (1962) of back pressuring during consolidation was employed. However, a back pressure of 1 Kg/sq cm was used rather than the recommended 2 Kg/sq cm. Further discussion of the back pressuring system and technique is given in the discussion of the testing procedure (Sec. 4.4).

A pore pressure reaction test was conducted immediately preceding the strength test. The procedure employed was as outlined by Dahlman (1965), with a few minor exceptions. Pore pressure readings were obtained by means of a pore pressure transducer and were taken over a period of 10 minutes, as outlined by Gilchrist (1967).

On completion of the pore pressure reaction test, the seating of the loading piston was checked and a strain dial was set up to measure vertical movement of the piston. Pore pressure and load cell readings were taken at increments of strain of 0.20% up to 3% strain, 0.50% up to 10% and 1.0% up to failure. As the tests progressed values of deviator stress, major and minor principal effective stress, effective principal stress ratio, and pore pressure parameter, $\bar{A}$, were calculated. These values and pore pressure were plotted versus percent strain.

Failure was considered to have occurred at maximum principal stress ratio. However, all tests were run until such time that a peak deviator stress was reached, except in cases where it was either impractical or beyond the limitations of the equipment. At the end of

the test all water lines leading to the sample were closed and the apparatus dismantled. Measurements taken at the end of the tests were final wet weights for moisture content determinations. In some cases final volume was measured by mercury immersion to provide a check on final degree of saturation. Typical data sheets and sample calculations for an undrained triaxial test are given in APPENDIX B.

CHAPTER IV

DISCUSSION OF SAMPLE PREPARATION AND TEST PROCEDURES

4.1 Soil Used in Program

The soil used in this program is a naturally occurring, surface-inactive soil. It was chosen because its mineralogy (little or no clay minerals) made this an ideal soil to start with. Subsequent modifications could readily be made by incorporating either different clay minerals into it, or different amounts of the same clay mineral.

4.2 Soil Modifications

The purpose of the soil modifications was to investigate the effect the addition of clay minerals had upon the energy absorption capacity and strength characteristics of the original soil.

The clay minerals chosen were a commercially prepared kaolin and a commercially prepared montmorillonite. As has been discussed, these modifying agents are for all practical purposes pure clay minerals. This fact, coupled with the fact that the original soil is essentially free of clay minerals, enables the study of the effects contributed independently by the clay minerals to energy absorption and strength characteristics of the soil to be more easily made. That is to say,

the effects contributed by one clay mineral are not masked by those contributed by another, as would be the case if more than one clay mineral were present. In order to compare and contrast the effects of different clay minerals, equal portions of kaolin and montmorillonite were used. Since montmorillonite is considerably more active than kaolin, selection of the proportion of modifying agent to be used was made on this basis. It was decided that 10 percent by weight montmorillonite would appreciably change the properties of the silt without excessively increasing the time required for consolidation. This was in fact found to be the case for the montmorillonite modification. However, the addition of 10 percent kaolin by weight did not significantly alter the index properties of the original silt. This fact, however, does not negate the usefulness of the test results in investigating the influence of kaolin on other properties of the soil. It merely confirms the low surface activity of the clay mineral kaolinite when compared to that of the clay mineral montmorillonite.

4.3 Moulding Triaxial Specimens

All triaxial samples were formed by consolidating the soil slurries in a one dimensional consolidation apparatus in four stages to a maximum vertical pressure of 1.50 Kg/sq cm. In order to ensure greater homogeneity, the samples were reversed and reloaded after primary consolidation had taken place under the final load increment. The samples were left in this state until such time as they were required for testing and in no case for less than two days. The similarity between samples of the same soil group can be estimated by comparing

the initial void ratios. These void ratios, including other pertinent results are listed in the Summary of Data Sheets in APPENDIX B. Although similar void ratios are not necessarily indicative of similar soil structures between specimens, the differences should be slight. Thus, the results available indicate that the moulding technique was satisfactory.

Early in the testing program, a moulding technique similar to that employed by Gilchrist (1967) was attempted. In this procedure 45 mm inside diameter consolidation tubes were used and the samples, after extrusion, were trimmed to an average diameter of 35.7 millimetres utilizing the Geonor trimming apparatus. This technique had to be abandoned because of the very dilatant nature of the silt. Even the slight disturbances present during the trimming process were sufficient to cause complete collapse of the sample.

In order to eliminate the problem of sample disturbance, the soil was moulded in consolidation tubes averaging 34 millimeters inside diameter. Since this size is very close to that obtained by trimming (35.7 mm), the samples could be trimmed to the desired length (80 mm) and extruded directly onto the pedestal of the triaxial cell. Vertical drainage strips could not be used inside the consolidation tubes to speed up the consolidation process, as they would tend to reduce the diameter of the sample and increase the amount of peripheral disturbance. In the course of sample preparation, it was found that vertical drains would have been desirable only for the montmorillonite modifications. Admittedly, some peripheral sample disturance does

take place with this technique also, but it is felt that it is of a minor nature, and does not significantly affect the test results.

4.4 The Triaxial Test

Drainage Aids

All triaxial specimens, after being extruded from the consolidation tubes were wrapped in saturated slotted filter paper and placed in contact with a bottom filter paper and porous stone. Internal drainage aids, such as wool wicks were not employed as it was felt that the insertion of the wicks would unduly disturb the sample. Without wicks in the sample the longest drainage path is approximately 1.7 centimetres. Under these drainage conditions the time required for consolidation was not found to be excessive.

The Load Cell, Pore Pressure Transducer and Volume Change Indicators

The same load cell as described by Gilchrist (1967) was used to measure loads in this testing program. Prior to use, the load cell was recalibrated on a Baldwin Type N, SR4 strain indicator. The sensitivity over the calibrated range was 0.033057 Kg/micro-inch/inch, which was sufficiently accurate to obtain the desired readings.

Pore pressure readings were obtained by means of a pore pressure transducer, which was also calibrated on the same strain indicator as the load cell. The sensitivity of the transducer over the calibrated range was 8.8521×10^{-4} Kg/sq cm/micro-inch/inch. In

the course of calibration it was found that the zero reading of the transducer was affected by the degree to which the transducer was tightened into the adaptor. In order to overcome this problem, the transducer was fitted to the adaptor under a torque of 20 inch-pounds, and was left connected to the adaptor throughout the entire testing program. The adaptor plus transducer was coupled to the triaxial cell by means of a slide valve assembly. Periodic checks on the zero reading showed it to be relatively stable.

Volume change measurements during consolidation were made with a twin-burette, constant volume change indicator which enabled back-pressuring during the consolidation process. With the back-pressure used (1 Kg/sq cm) this is a desirable feature, as a greater time can be allowed for air to go into solution. Another feature of these volume change indicators is that the burettes have a 5 millilitre capacity, and are therefore quite sensitive to volume change. This is essential as the soils employed show very little tendency to rebound and it would be very difficult to ascertain when rebound is complete with the conventional stop-cock burettes. Calculations made using the results from the volume change indicators were on several occasions checked with final moisture contents, and in all cases the agreement was good. This tends to confirm the accuracy of the volume change measurements.

Back-Pressure, Pore-Pressure Reaction and Strain Rate

Prior to consolidation of the triaxial specimen, a back pressure of 1 Kg/sq cm was applied while simultaneously increasing the

cell pressure also by 1 Kg/sq cm. The purpose of back pressuring is to dissolve any entrapped air, and thereby eliminate its effects on pore pressure reaction and volume change measurements. In order to dissolve entrapped air Bishop and Henkel (1962) recommend the use of a back pressure of approximately 2 Kg/sq cm. However, in order to retain a reasonably broad range of consolidation pressures (2 to 8 Kg/sq cm) and still remain within the capacity of the Geonor equipment a back pressure of 1 Kg/sq cm was selected. It was felt that with this back pressure and back pressuring during the consolidation process the entrapped air would be dissolved and would not adversely affect measurements of volume change and pore pressure.

Pore pressure reaction tests were performed by instantaneously increasing the cell pressure by 1 Kg/sq cm with all drainage lines closed, and measuring the build up of pore pressure over a 10 minute interval. The results of the pore pressure reaction tests are summarized in TABLE IV.1.

TABLE IV.1

SUMMARY OF PORE PRESSURE REACTION TESTS

Confining Pressure σ_3 (Kg/cm^2)	$\Delta\sigma_3$ (Kg/cm^2)	$\Delta\sigma_3$ % of σ_3	Initial Degree of Saturation %			Pore Pressure Reaction Over 10 Min. %		
			Natural Soil	Kaolin Modification	Montmorillonite Modification	Natural Soil	Kaolin Modification	Montmorillonite Modification
2 N/C	1	50	100.0	92.5	96.0	87	91	73
4 N/C	1	25	100,98.1,100	95.6	96.6	36,38,44	61	76
6 N/C	1	16.7	99.4	93.6	91.2	66	61	46
8 N/C	1	12.5	100.0	94.5	96.0,96.4	66	64	68,35
2 O/C	1	50	98.3	95.1	97.5	68	76	55
4 O/C	1	25	100.0	93.5	96.9	60	80	26
6 O/C	1	16.7	100.0	94.8,94.4	97.6	33	40	27,31
4 N/C*	1	25	-	-	96.2	-	-	37
4 N/C	2	50	-	-	96.2	-	-	47
4 N/C	3	75	-	-	96.2	-	-	54
4 N/C	4	100	-	-	96.2	-	-	64

N/C Normally consolidated samples

O/C Over consolidated samples

* Special test series

It is evident that in most cases the reactions are extremely low. Since a pore pressure transducer connected directly to the base of the cell was used to measure pore pressure, expansion of drainage lines is not a contributory factor to low pore pressures, as may be the case when a null indicator is employed. In nearly all cases the initial degree of saturation is high, and where final degrees of saturation were computed they were found to be very close to 100 percent. It would therefore appear that the low pore pressures cannot be attributed to large quantities of undissolved air in the specimens. It is also unlikely that large quantities of undissolved air were trapped in the pore pressure lines and measuring system as considerable care was taken in de-airing the system prior to and after consolidation of the specimen. It is the author's opinion that the low pore pressure reactions can be attributed to a combination of a number of factors.

Whitman and Richardson (1961) in their work on time-lags in pore pressure measurements state that a mineral skeleton of low compressibility approaching that of water can account for low reactions. They also state that interparticle bonding, structural viscosity, and very tight packing of particles are reasons why the stiffness of the mineral skeleton might approach that of water. Of these factors it appears that the latter two may have some applicability to these specimens. For the natural silt and kaolin modification the high strengths obtained tend to suggest that these soils may be fairly tightly packed and would thus tend to exhibit considerable rigidity. It is not known whether the rigidity is approaching the compressibility

of water or not. However, there is evidence that this may be a contributing factor to low pore pressure reactions for the silt and kaolin modification. In the case of the montmorillonite modification even lower pore pressure reactions were obtained. This seems to be consistent with the findings of Thomson (1963), Locker (1963), Dahlmann (1965) and Gilchrist (1967). These individuals found that very low pore pressure reactions were obtained for sodium modified clays. Since the montmorillonite used was primarily a sodium montmorillonite, it seems understandable that low pore pressure reactions were obtained in this series of tests. It is the author's opinion that the low reactions can be attributed to structural viscosity. The very large water hulls that surround sodium modified clay particles tend to behave in a somewhat viscous fashion, and as a result would tend to impart a greater degree of viscosity to the entire soil skeleton. Whitman and Richardson also point out that the influence of the preceding factors on pore pressure reactions are greatest when remoulded samples and low pressure increment ratios are used.

In the case of this series of tests all soil samples were remoulded. It is also of interest to note from TABLE IV.1 that a special series of pore pressure reaction tests was run on a normally consolidated sample of montmorillonite modified silt. It is evident that as the pressure increment ratios increased the pore pressure reactions also increased. This same trend is also found for other samples, but is much less apparent. Thus it seems that the findings here are consistent with those of Whitman and Richardson.

The problem of low pore pressure reactions could likely have been alleviated to some extent by using a higher back pressure, higher pressure ratio increments, and placing internal drains in the specimen. Schmertmann and Osterberg (1960) found that the use of wool wicks in the specimen and also in direct contact with the bottom cap insures a very rapid transmission of pore pressure throughout the entire sample. The remedial measures just mentioned, however, were not employed due to the sensitivity of the specimens as has been discussed.

Bishop and Henkel (1962) state that for the undrained triaxial test with pore pressure measurements it is generally desireable to run the test relatively slowly in order to ensure uniformity of pore pressure throughout the sample. A strain rate of approximately 2.25 percent strain per hour was selected for the entire testing program. A comparison of this strain rate with the values suggested by Bishop and Henkel (1962) for various soils, indicates that it is adequate for the soils tested. An added feature of this strain rate was that specimen failure was achieved within a normal working day.

<u>Failure Criterion</u>

The maximum deviator stress and the maximum principal effective stress ratio are the two failure criteria generally adopted for triaxial testing of cohesive soils. Bjerrum and Simons (1960) state:

> "The pore pressure at the point of maximum deviator stress either attains a maximum value and the two failure criteria coincide, or it is still increasing and only reaches a maximum value upon further strain. If the deviator stress

> is constant or only decreases slightly with strain after its maximum value, then the maximum principal effective stress ratio point occurs after the point of maximum deviator stress."

The latter of these two pore pressure conditions was found to be the condition that existed for the montmorillonite modified silt, and the maximum principal stress ratio occurred after the maximum deviator stress. However, for the unmodified silt and the kaolin modifications the pore pressure generally attained a maximum value before either the maximum deviator stress or maximum principal effective stress ratio. It was also found that for the two soil types the maximum principal effective stress ratio occurred prior to the maximum deviator stress, which generally occurred at strains in the order of 16 to 20 percent. It is felt that the very dilatant nature of the original soil and the kaolin modification is responsible for this behaviour. Under small deviator stress the soil begins to dilate and the pore pressure correspondingly drops off.

The maximum principal effective stress ratio was chosen as the failure criterion for all soil types. It was felt that it was more meaningful for the unmodified silt and kaolin modification, in that severe sample distortion at high strains reduces the validity of calculations of stress at these strains. For the montmorillonite modified silt either of the strength criteria appear to be equally applicable. However, selection of the effective stress ratio as the failure criterion results in a better basis for comparison of results between the three soils. In all three cases the maximum principal effective stress ratios generally occur at strains in the order of 8 to 10 percent.

CHAPTER V

PRESENTATION OF RESULTS

5.1 Introduction

The purpose of this thesis was to provide additional information concerning any relationships that may exist between soil strength characteristics and energy applied, stored, or liberated, during the consolidation processes. Energy was imparted to the soil samples by means of isotropic consolidation in the triaxial cell, and strength characteristics were obtained from undrained triaxial compression tests with pore pressure measurements. The results of the isotropic consolidation and undrained compression tests are presented in this chapter. The interpretation and discussion of these results is given in Chapter VI, following.

5.2 Computed Strength Parameters

Measurements of pore pressure and axial load were made at increments of strain of 0.20% to 3% strain, 0.50% to 10%, and 1.0% to failure. As the tests progressed values of deviator stress, major and minor principal effective stress, effective principal stress ratio, and pore pressure parameter, $\bar{A}$, were calculated. A typical set of complete data sheets and calculations is given in APPENDIX C. The

above parameters, including pore pressure were plotted versus percent strain, and typical curves are presented in FIGURES V.1 to V.4. The remainder of the curves including other pertinent results are included in APPENDIX D.

In order to compare the original soil and the modifications a common confining pressure of 6 Kg/sq cm was arbitrarily chosen. Although the samples did not have the same final void ratio, it is evident that certain comparisons can still be made.

FIGURE V.1 is a plot of deviator stress versus axial compressive strain. From this plot it is evident that the curve for the kaolin modification has the same typical shape as that for the unmodified silt. In both cases the deviator stress rises quickly with strain to a value in the order of 2 percent. Beyond this value the increase in strength is more gradual, but is nevertheless appreciable and continues to a peak value at a strain in the order of 15 to 16 percent. It is felt that this behaviour can be attributed to the very dilatant nature of the silt and the kaolin modification. In the case of the montmorillonite modification the deviator stress tends to peak at a value of strain in the order of 2 percent, and then shows a continuous but gradual drop in magnitude. As noted previously, in order to draw a more valid comparison between strengths for the three soil types the principal effective stress ratio was used as a failure criterion.

FIGURE V.2 is a plot of the principal effective stress ratio versus the axial compressive strain. It can be seen that all three

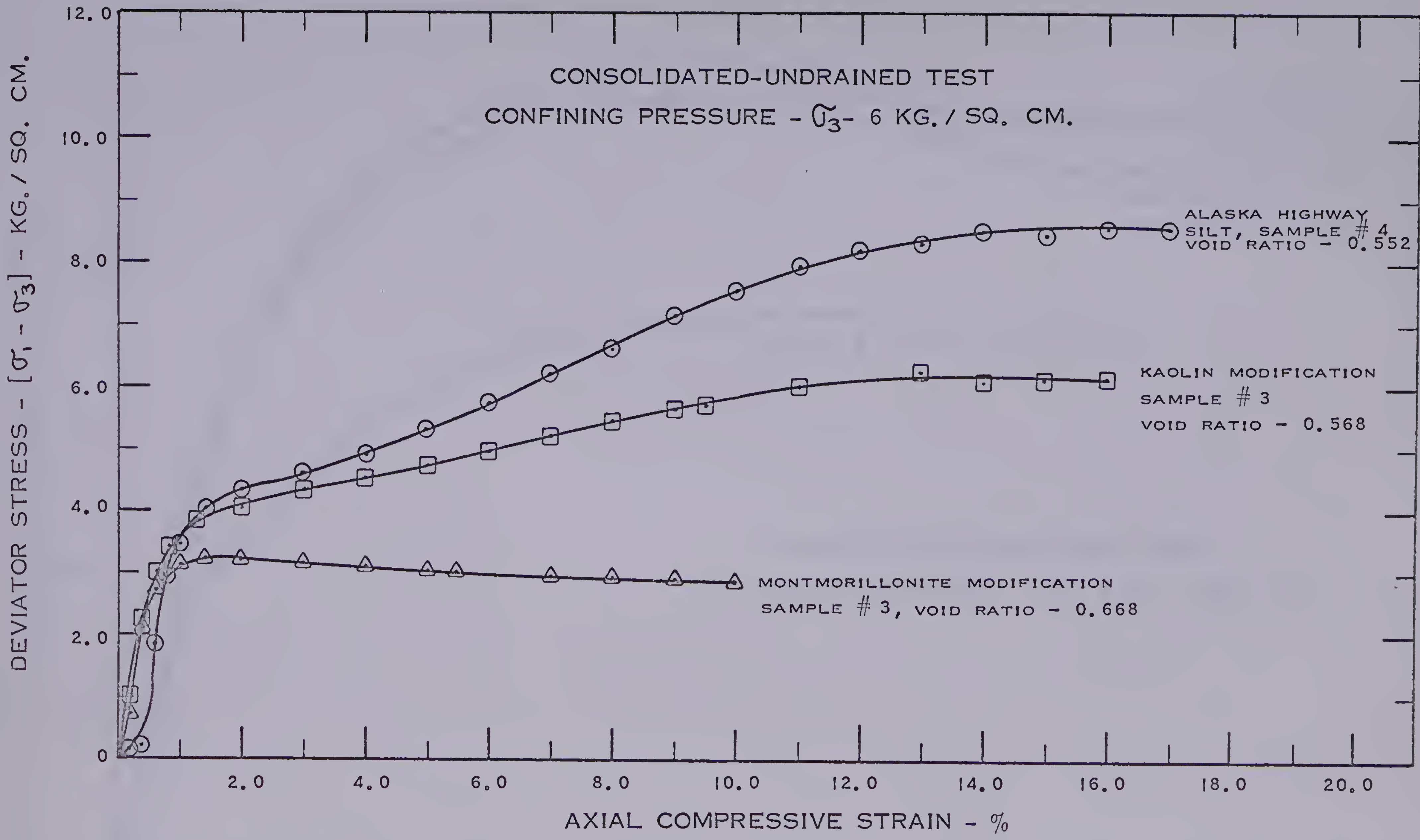

FIGURE V.1 DEVIATOR STRESS VERSUS AXIAL COMPRESSIVE STRAIN

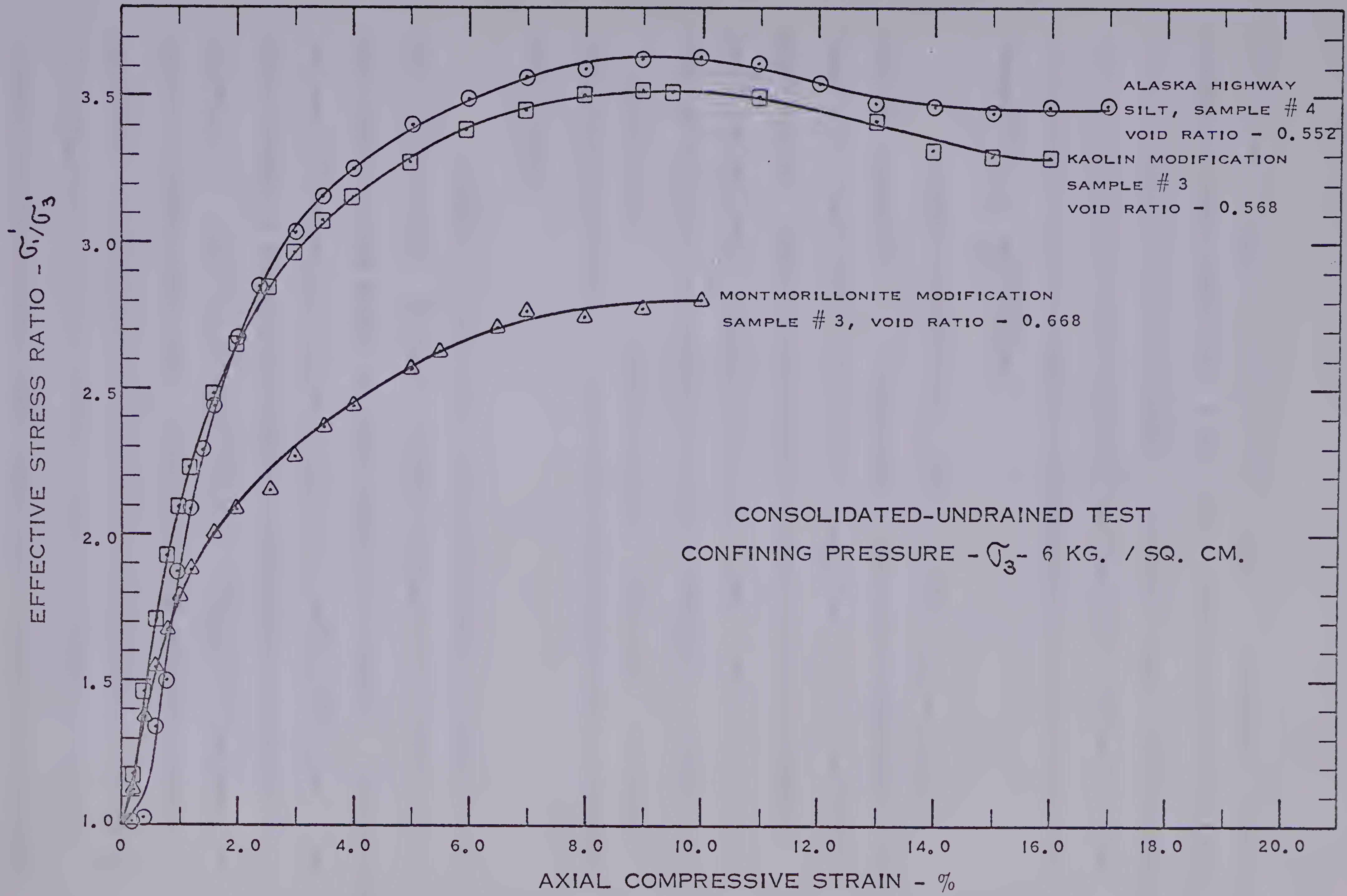

FIGURE V.2 EFFECTIVE STRESS RATIO VERSUS AXIAL COMPRESSIVE STRAIN

curves tend to peak at a strain in the order of 10 percent. This fact, as mentioned previously (Sec. 4.4), tends to substantiate the selection of the strength criterion employed. It is also evident that the curves for the silt and the kaolin modifications are again similar in shape, and that they differ in magnitude substantially from the curve for the montmorillonite modification.

FIGURES V.3 and V.4 are plots of pore pressure and pore pressure parameter, $\bar{A}$, respectively, versus axial compressive strain. These plots again show a similarity in shape for the silt and kaolin modification. They also tend to confirm that the silt and kaolin modifications tend to exhibit a dilatant nature. The rapid rise in pore pressure and pore pressure parameter followed by a gradual decrease is characteristic of a dilatant structure. The curves for the montmorillonite modification, on the other hand, tend to show no tendency for dilation.

FIGURES V.5, V.6 and V.7 are modified Mohr diagrams for the three soils tested. In order to obtain the best fit envelopes for the data available, the method of least squares was employed. By this means the best fit curves are obtained without bias being introduced. From the envelopes so obtained the effective peak angles of shearing resistance, ϕ' , were computed to be 34.1, 33.6 and 27.5 degrees for the silt, kaolin modification, and montmorillonite modification respectively. Although cohesion intercepts were computed in all three cases, they are, for all practical purposes, negligible. It is of interest to note the considerable reduction in the angle of shearing

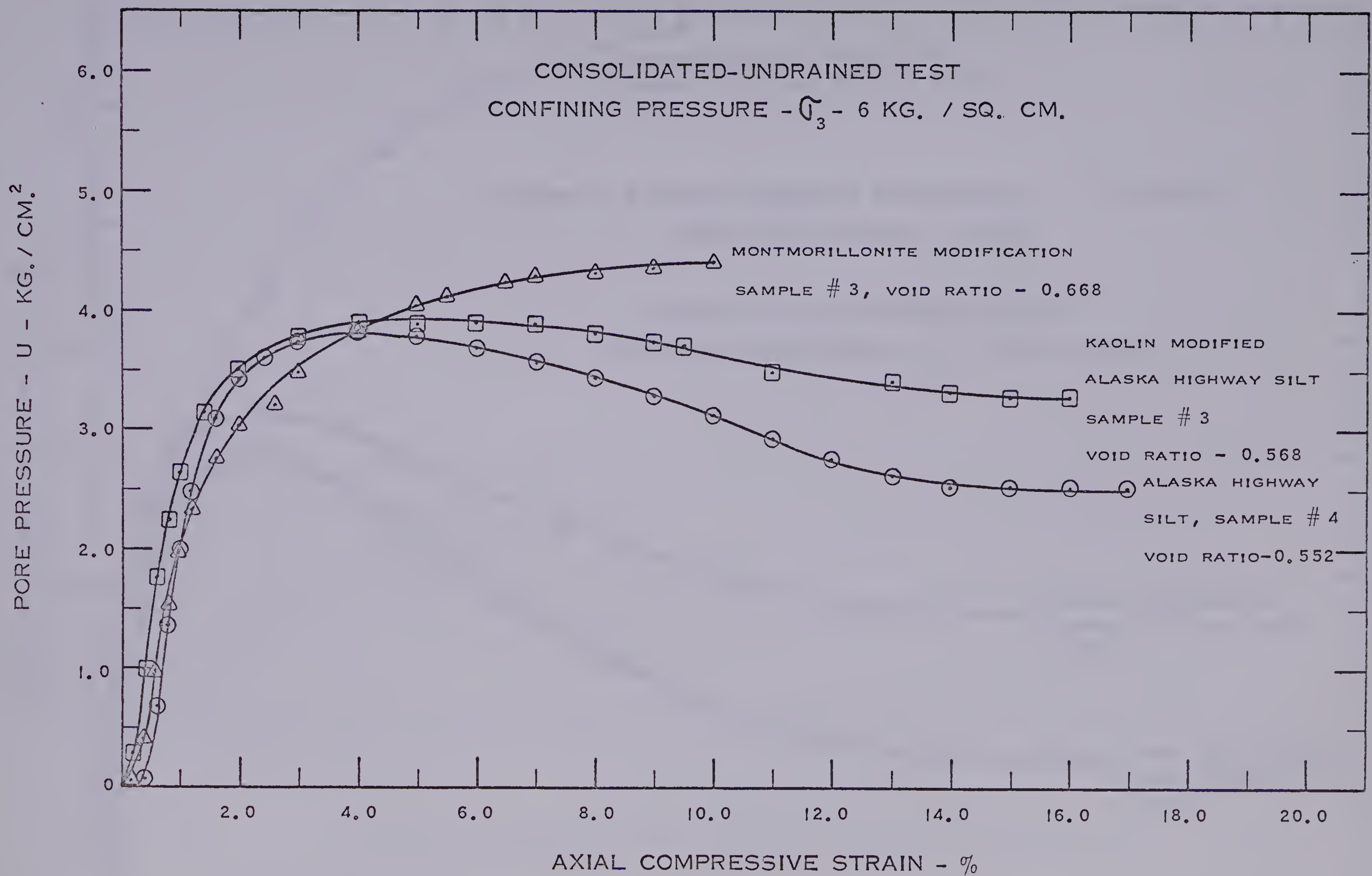

FIGURE V.3 PORE PRESSURE VERSUS AXIAL COMPRESSIVE STRAIN

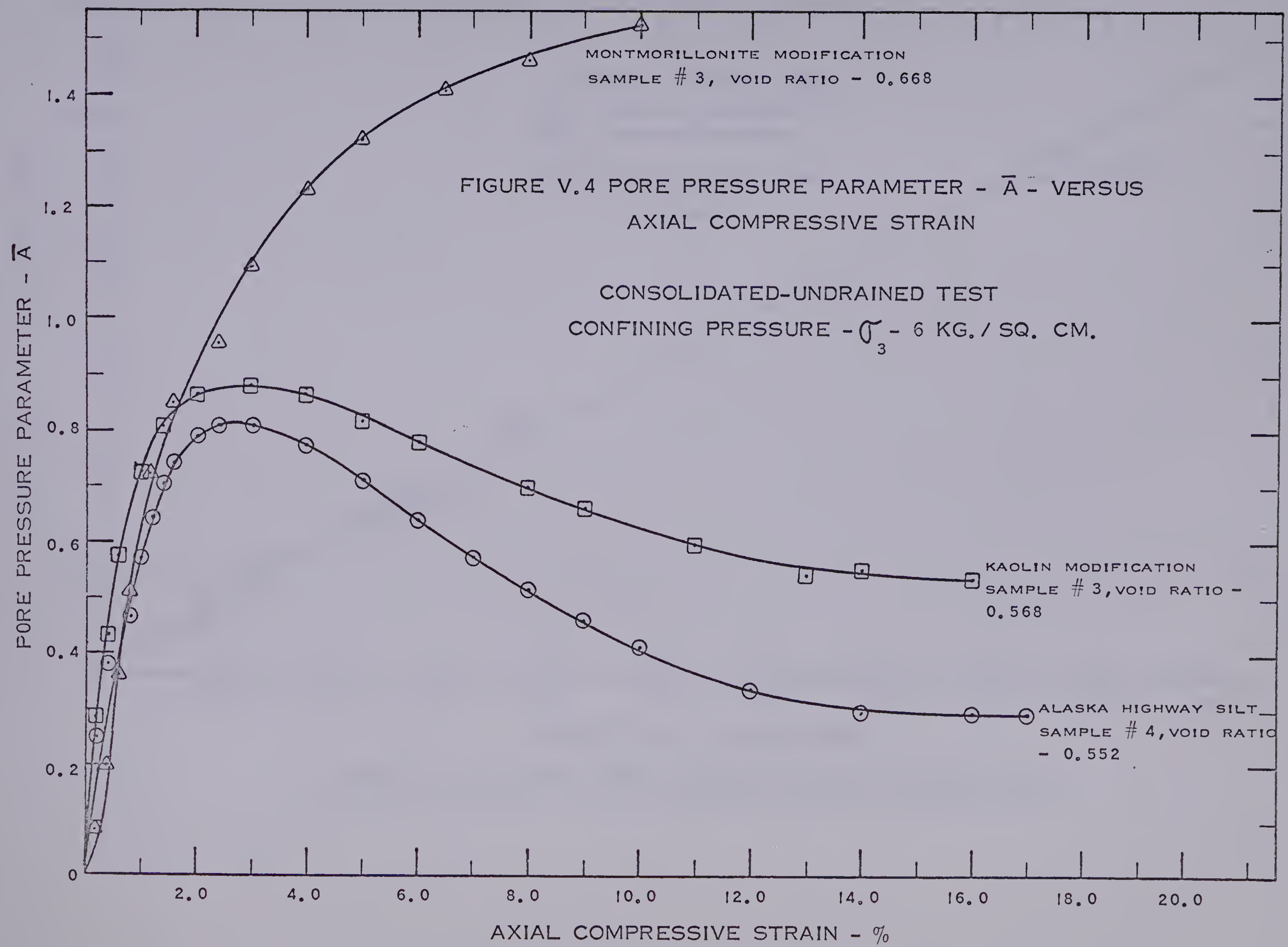

FIGURE V.4 PORE PRESSURE PARAMETER - $\bar{A}$ - VERSUS AXIAL COMPRESSIVE STRAIN

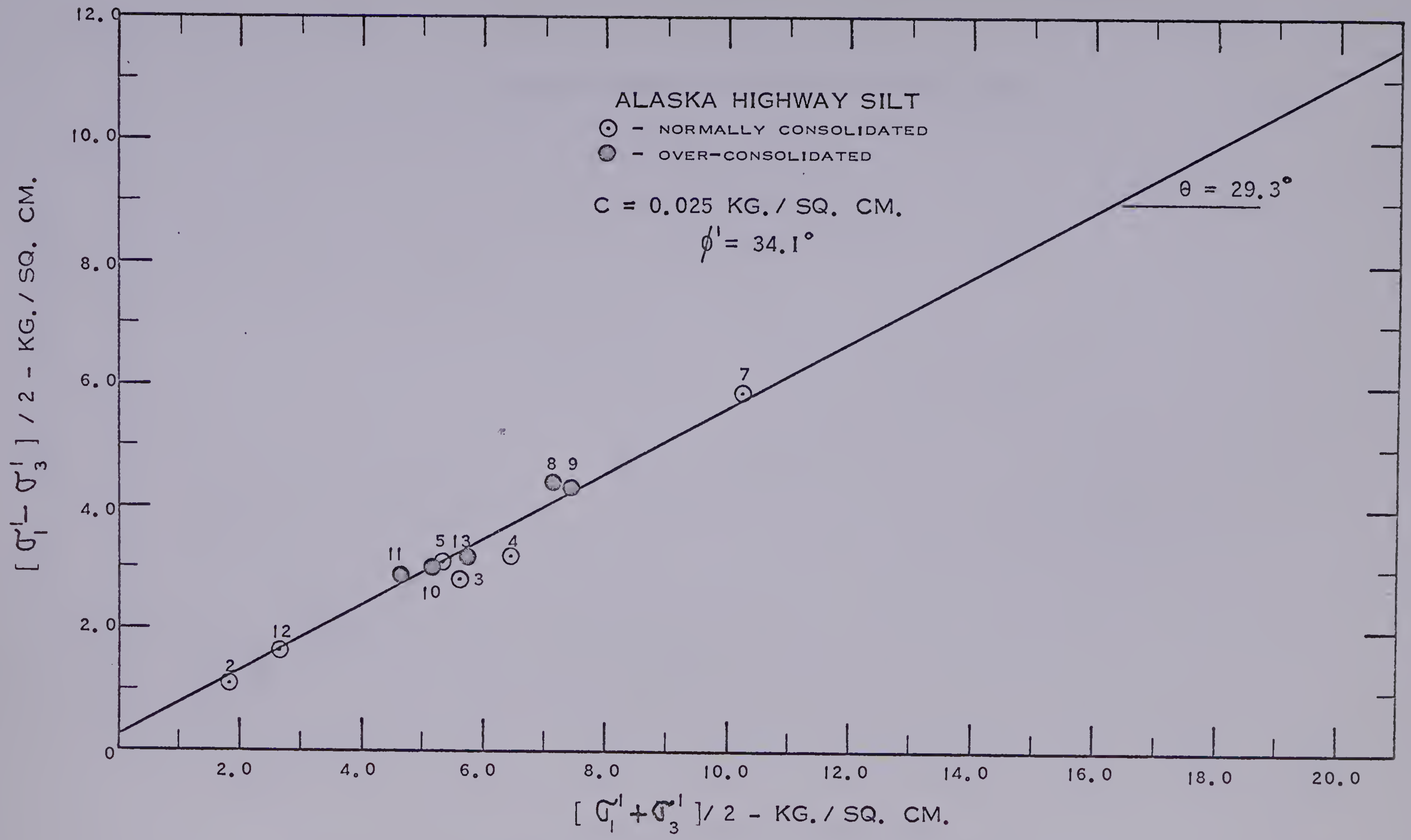

FIGURE V.5 MODIFIED MOHR DIAGRAM, ALASKA HIGHWAY SILT

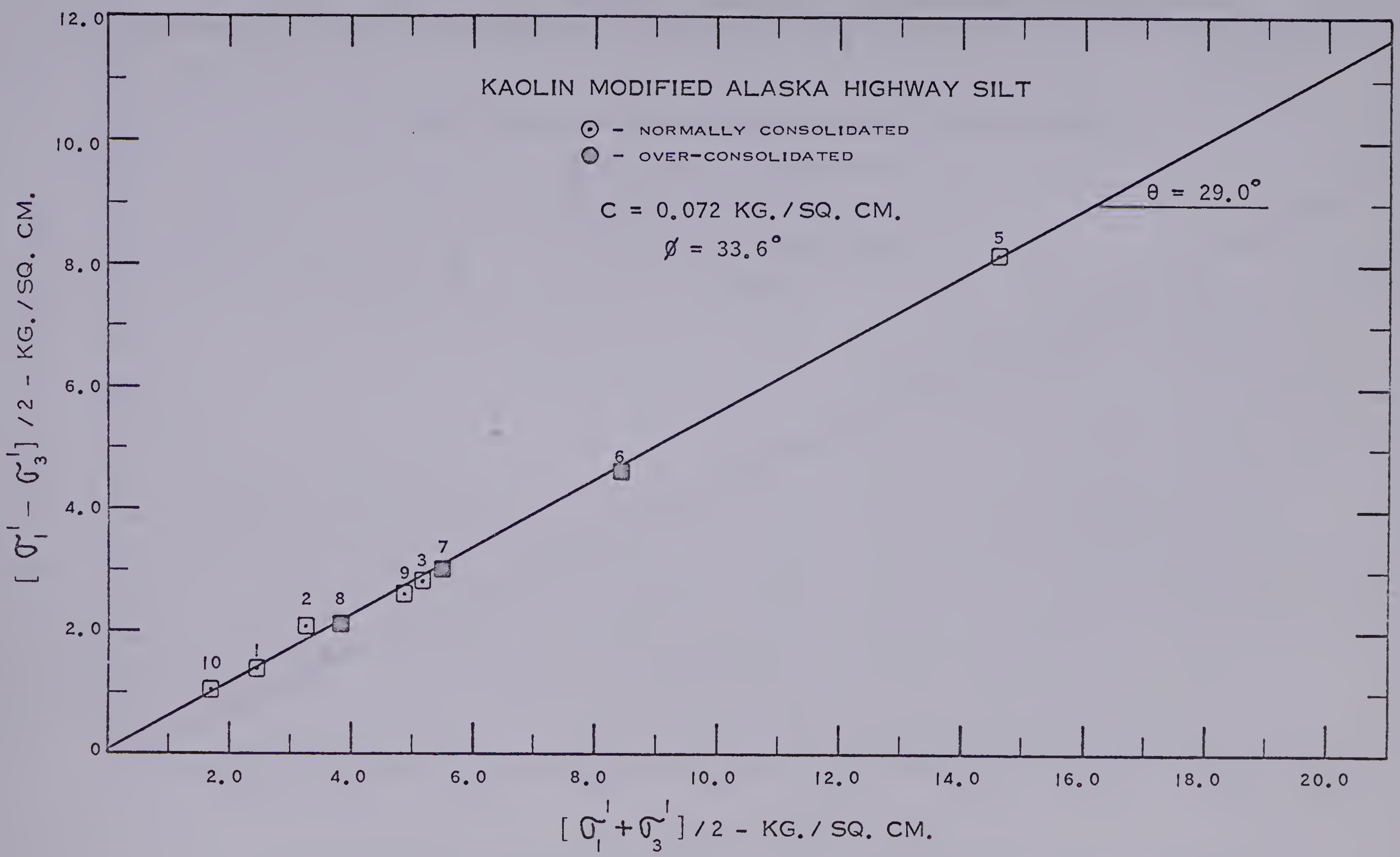

FIGURE V.6 MODIFIED MOHR DIAGRAM, KAOLIN MODIFIED SILT

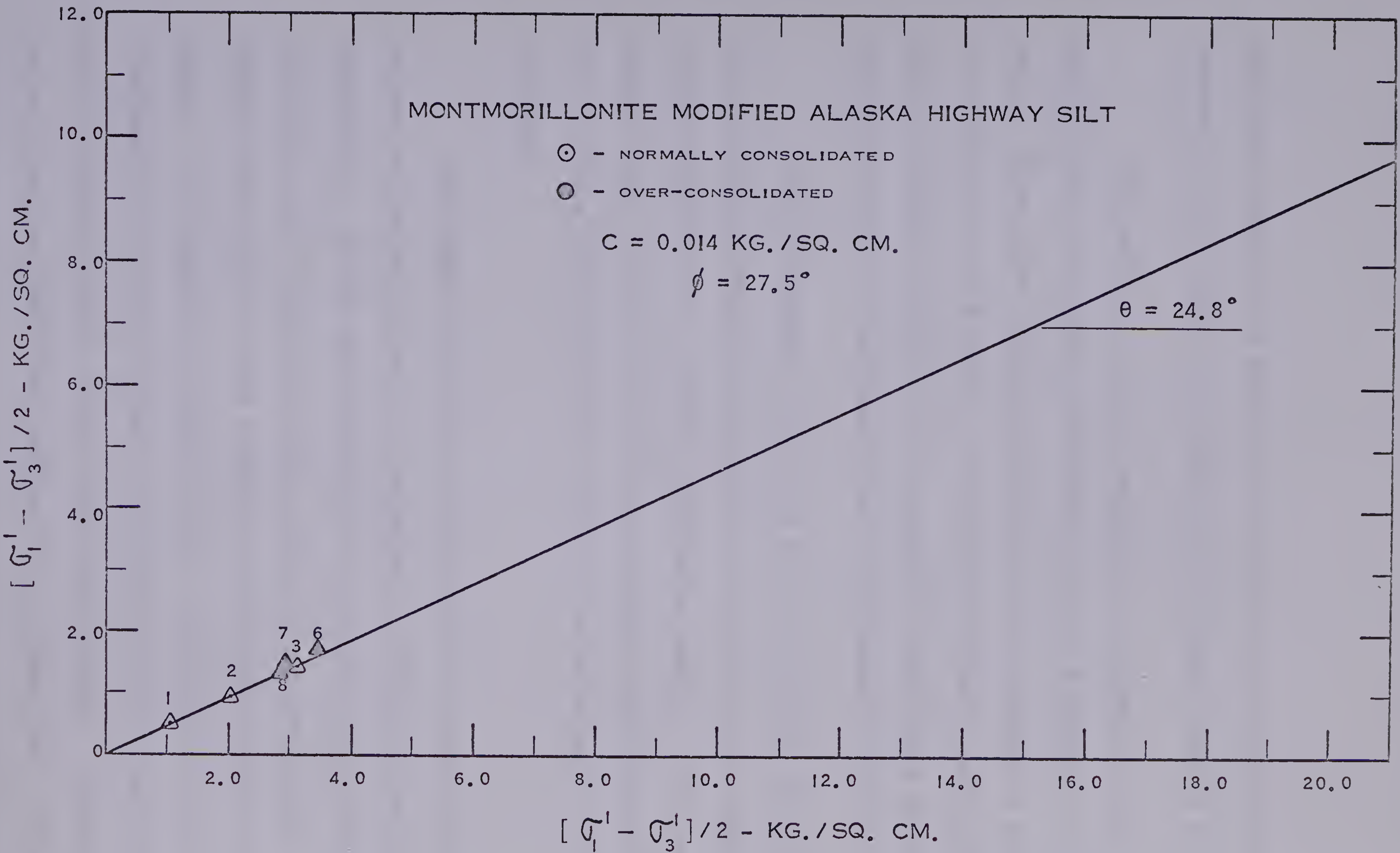

FIGURE V.7 MODIFIED MOHR DIAGRAM, MONTMORILLONITE MODIFIED SILT

resistance caused by the addition of 10 percent montmorillonite as compared to that caused by the addition of the same quantity of kaolin.

The addition of kaolin resulted in a decrease in the effective angle of shearing resistance of only 0.5 degrees or 1.5 percent, whereas the addition of montmorillonite resulted in a decrease of 6.6 degrees or 19.5 percent. This tends to show the very active nature of montmorillonite, as it is generally found that a decrease in the angle of shearing resistance occurs with increasing plasticity.

Also of interest is the fact that in all cases the test results for the over-consolidated samples tend to fall on the same envelope as those for the normally consolidated samples. This tends to indicate that, for the soils and test conditions used, over-consolidation or stress history has little influence on the angle of shearing resistance.

Semi-logarithmic plots of final void ratio versus confining pressure and deviator stress are presented in FIGURES V.8 and V.9, respectively. For the data presented it is evident that linear relationships exist between void ratio, and confining pressure and deviator stress, both for the normally consolidated and over-consolidated samples. It is generally accepted (Taylor, 1948) that this type of relationship is valid for normally consolidated samples, and that it may not necessarily apply to over-consolidated samples. However, for the soils and test conditions employed it is evident that a linear relationship appears to exist also for the over-consolidated samples. As a result straight lines were fitted to the data.

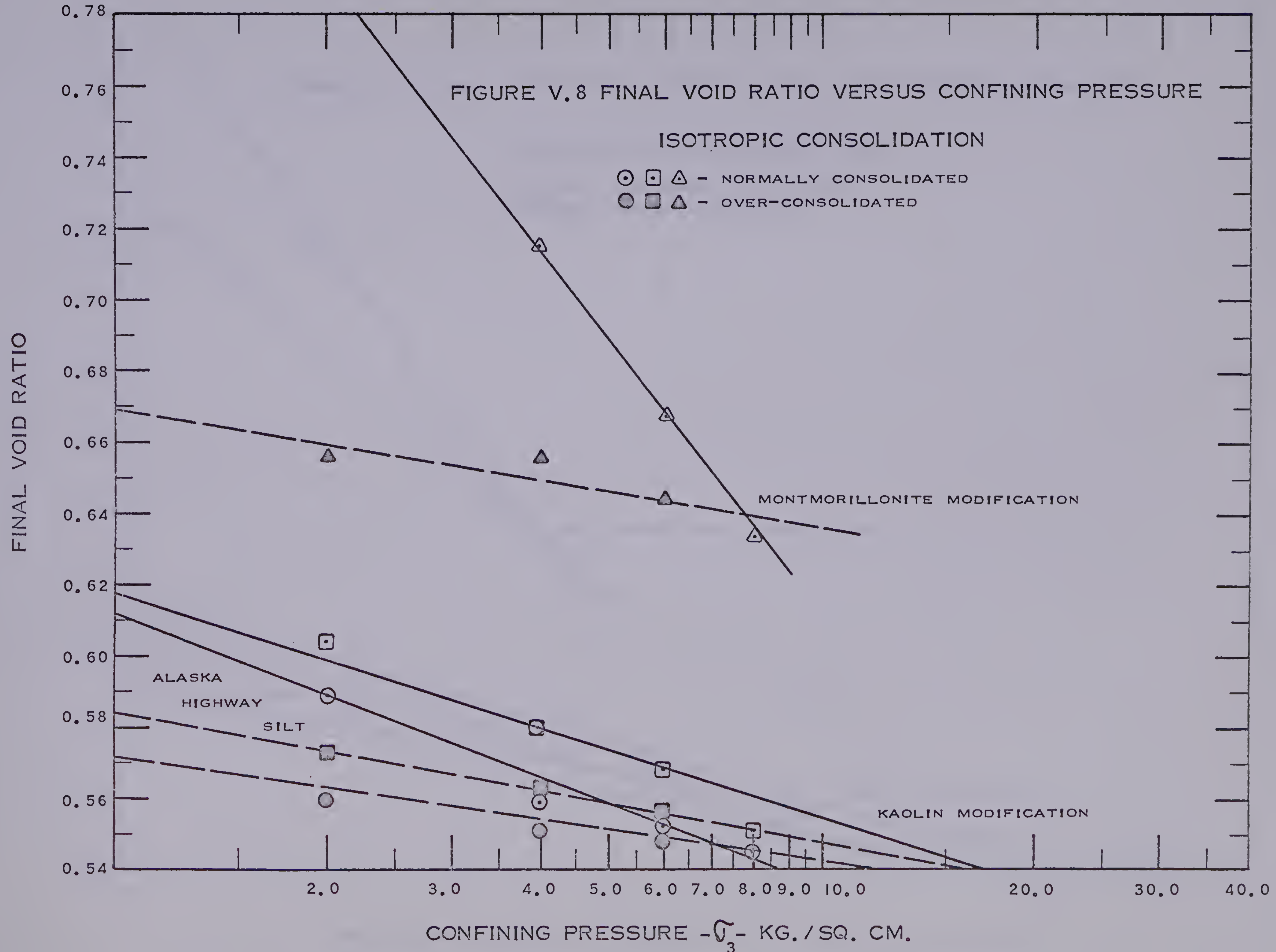

FIGURE V.8 FINAL VOID RATIO VERSUS CONFINING PRESSURE

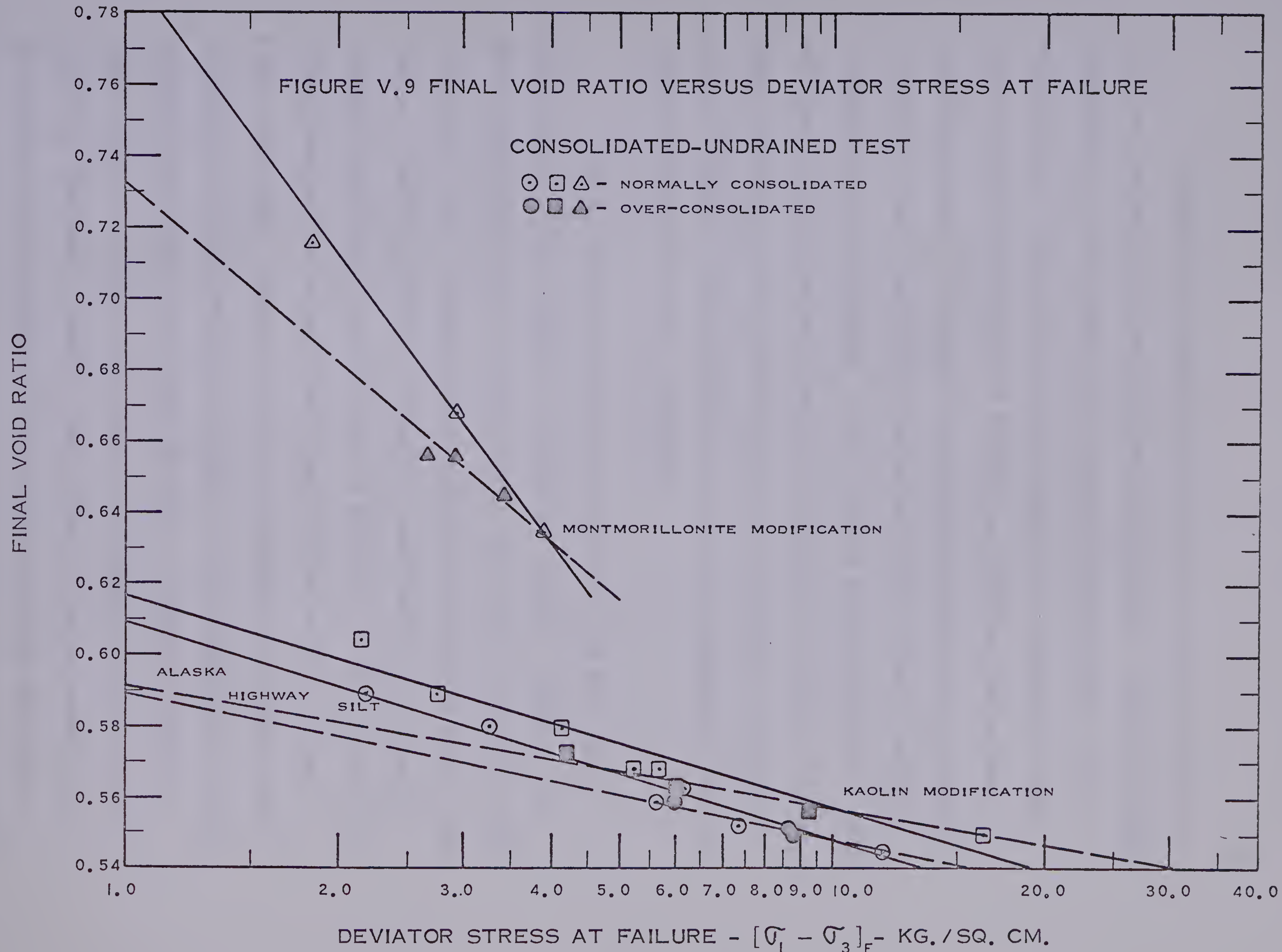
FIGURE V.9 FINAL VOID RATIO VERSUS DEVIATOR STRESS AT FAILURE
CONSOLIDATED-UNDRAINED TEST
- NORMALLY CONSOLIDATED
- OVER-CONSOLIDATED
MONTMORILLONITE MODIFICATION
KAOLIN MODIFICATION
ALASKA
HIGHWAY
SILT
FINAL VOID RATIO
0.78
0.76
0.74
0.72
0.70
0.68
0.66
0.64
0.62
0.60
0.58
0.56
0.54
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
20.0
30.0
40.0
DEVIATOR STRESS AT FAILURE - $[\sigma_1 - \sigma_3]_F$ - KG./SQ. CM.

In order to eliminate any bias in the selection of the best fit line, the method of least squares was employed. Lines were fitted independently to the normally and over-consolidated points. As a result of this fitting procedure the lines do not necessarily intersect at a confining pressure of 8 Kg/sq cm, which was the maximum confining pressure for normally consolidated samples and the pressure to which all over-consolidated samples were taken prior to rebound. If both figures are compared, it will also be apparent that for a given soil type the lines do not intersect at a common void ratio. However, other than for the kaolin modification, the discrepancy is small and can be disregarded. More will be said about the treatment of the discrepancy for kaolin in later paragraphs.

The similarity between the natural silt and the kaolin modification is again apparent in these two figures. However, the kaolin modification has a slightly higher void ratio and strength than the natural silt. The approximate parallelism between the curves shows that as a confining pressure is applied consolidation and strength increase take place at the same rate in both soils. For a given confining pressure the montmorillonite modification exists at a much higher void ratio. However, the rate of decrease of void ratio with increasing confining pressure is greater for the montmorillonite modification than either of the other two soils. FIGURE V.9 tends to suggest that, for a given void ratio, the strength increases as the plasticity increases. However, there appears to be an anomaly here as the curve for the montmorillonite crosses the other two curves. Perhaps, with more data, the curves would actually approach each other

at some minimum void ratio, and the compression process would no longer tend to be the governing process in strength increase. That is to say, no further volume change, regardless of the magnitude of the confining pressure could take place without physically compressing the particles themselves.

The curves of FIGURES V.8 and V.9 form a basis for the relationships presented between energy as applied in the consolidation process and the strength characteristics of the soils tested.

5.3 Energy-Strength Relationships

FIGURE V.10 has been derived directly from FIGURE V.8 and is, in fact, the same plot except that it has been plotted on an arithmetic scale. The initial or "zero" void ratios for the normally consolidated branches have been taken as the averages of all initial void ratios prior to consolidation. The void ratios that would exist for the over-consolidated case and zero confining pressure have been found simply by extrapolation of the over-consolidated branch of the curve to zero.

It has been shown in section 2.2 that curves of this nature are similar to the stress-strain curves derived by Brooker (1967) for anisotropic consolidation of several naturally occurring remoulded soils. It has also been shown that the areas bounded by these curves have units of Kg cm/cu cm which are units of energy per unit volume. On this basis the area beneath the normally consolidated branches was considered to be the energy applied in the consolidation process.

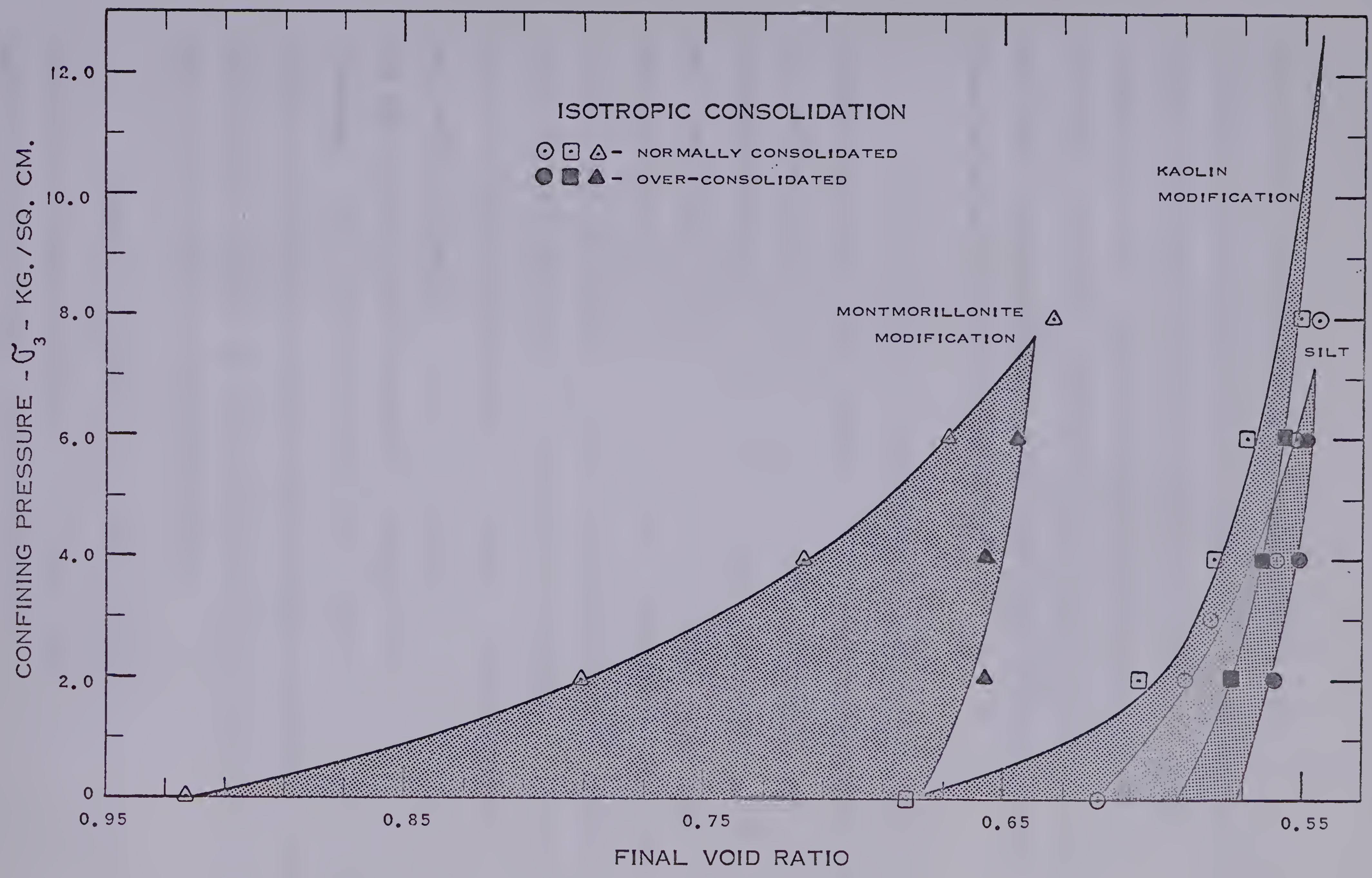

FIGURE V.10 FINAL VOID RATIO VERSUS CONFINING PRESSURE

Similarily the area beneath the over-consolidated branch is the energy liberated, and the energy within the two curves is the energy stored. Energy computations were made at arbitrarily chosen and equally spaced void ratios. By this means and with the aid of FIGURE V.9 the energy required in consolidating a particular sample to a given void ratio, and the strength at that void ratio could be obtained. These values were plotted in FIGURE V.11.

It is of interest to note that for the normally consolidated samples in all cases there appears to be a linear relationship between the deviator stress at failure and the energy that was applied in consolidating the sample. This does not appear to be the case for the over-consolidated branch. The relationship between the energy liberated and the deviator stress obtained for over-consolidated samples is non-linear and appears to become increasingly so with increasing plasticity, assuming that the interpolation indicated by the dashed line for the kaolin modification is correct. This would appear to be a valid assumption in view of the behaviour of the other soil types, and the fact that the discrepancy is brought about by the fitting procedure. It is felt that had the test results for the kaolin modification exhibited less scatter, less discrepancy would have resulted. Unfortunately, neither the time nor the additional samples were available to run a series of confirmation tests.

FIGURES V.12 and V.13 are plots of the effective angle of shearing resistance and the plasticity index respectively, versus applied and stored energy. Although the results are limited it is nevertheless evident that certain trends exist.

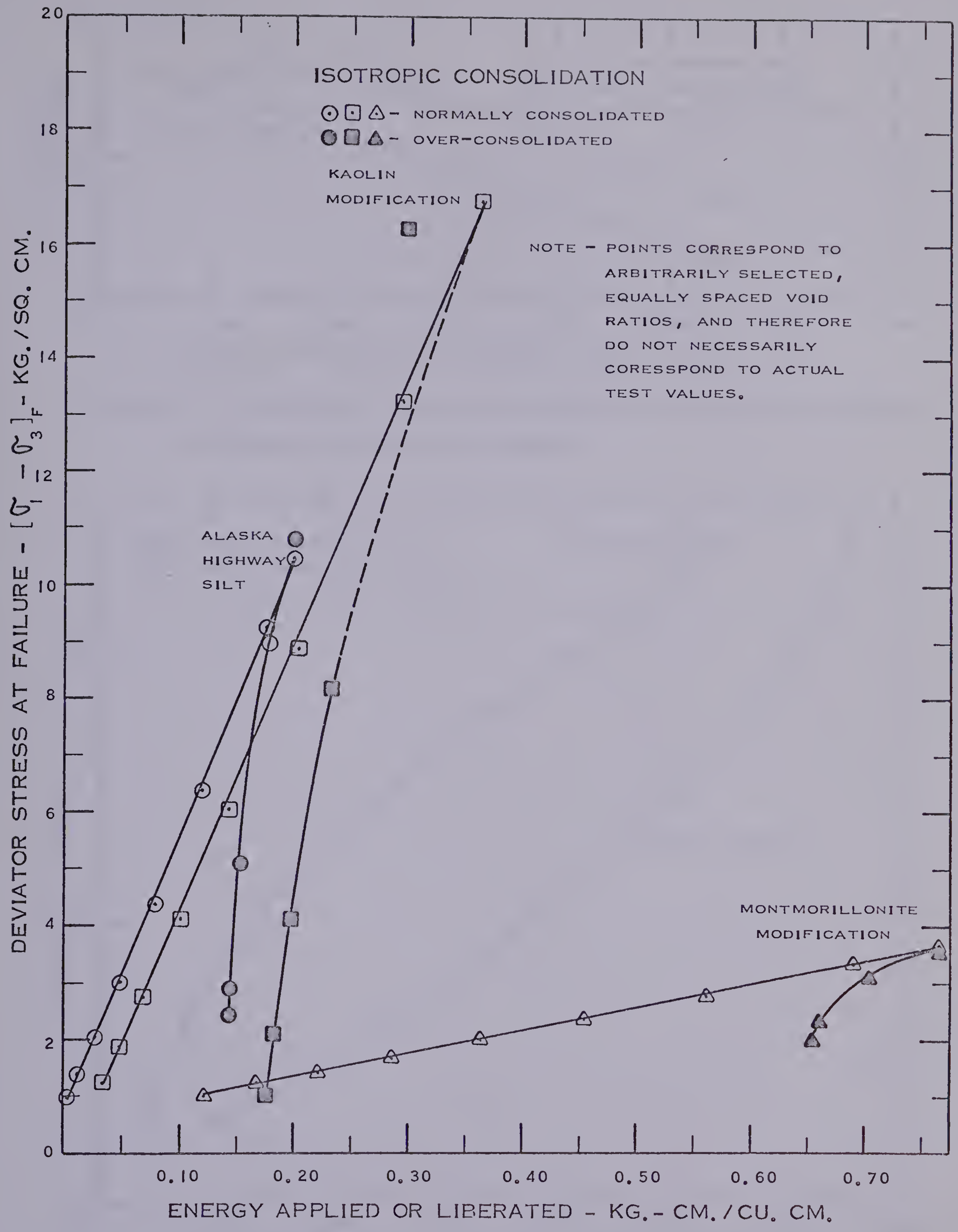

FIGURE V.II DEVIATOR STRESS AT FAILURE VERSUS ENERGY APPLIED OR LIBERATED

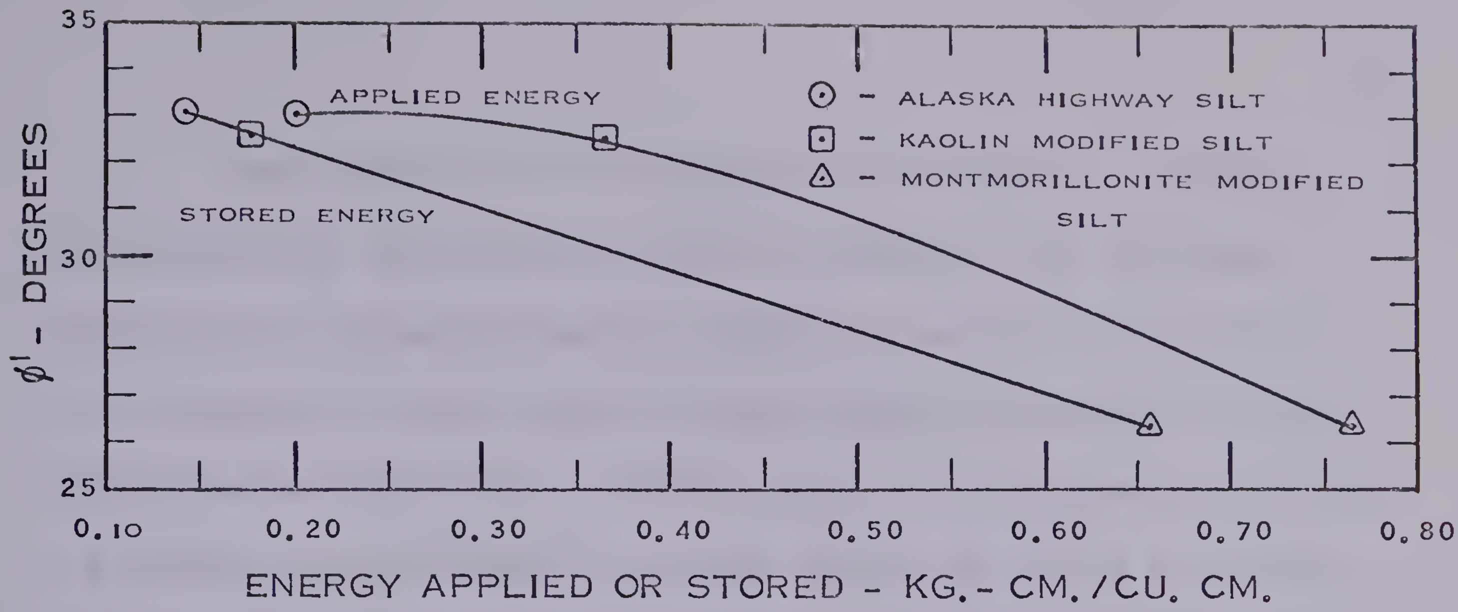

FIGURE V.12 EFFECTIVE ANGLE OF SHEARING RESISTANCE VERSUS ENERGY APPLIED OR STORED

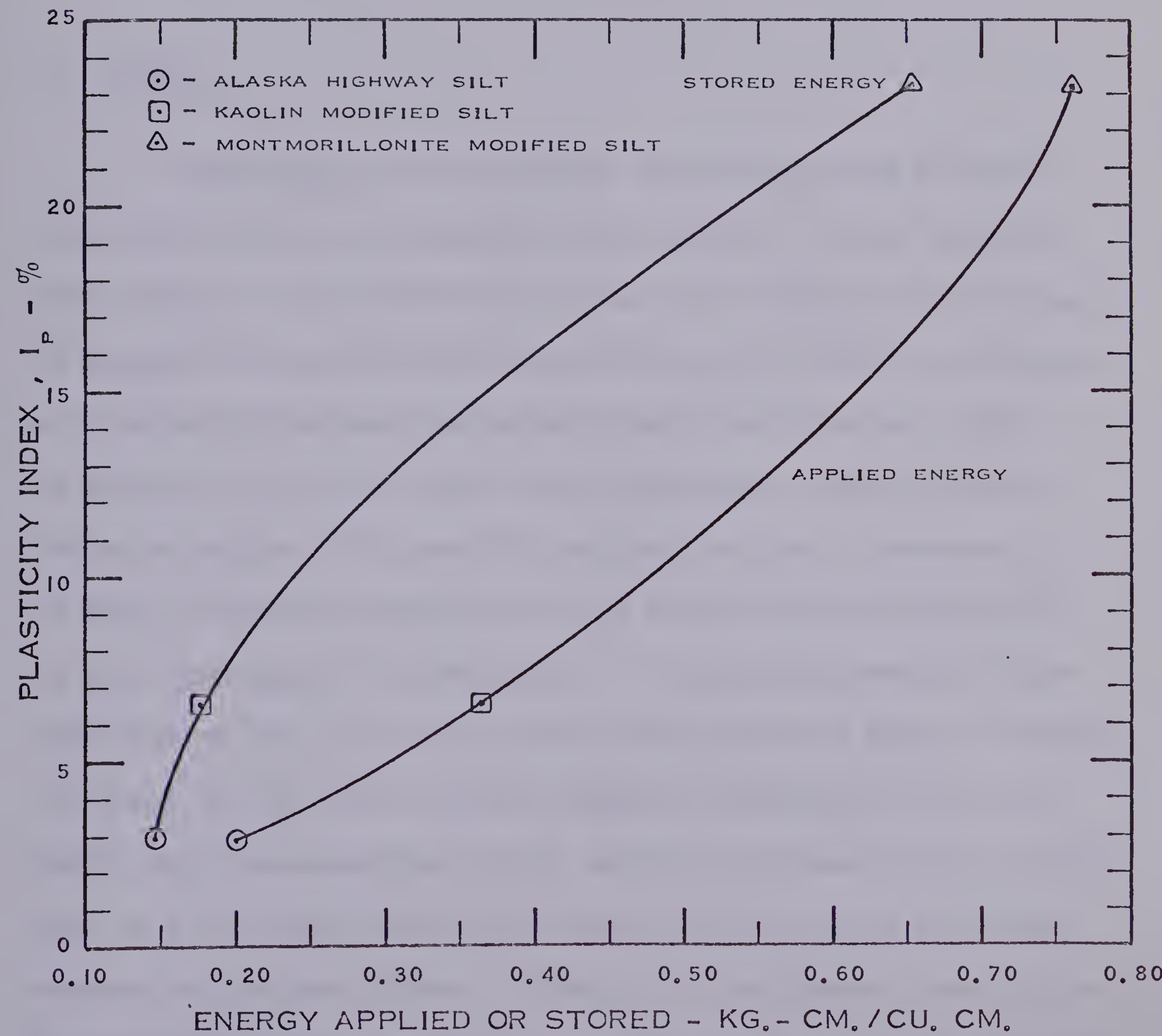

FIGURE V.13 PLASTICITY INDEX VERSUS ENERGY APPLIED OR STORED

From FIGURE V.12 it is apparent that there is a tendency for the angle of shearing resistance to decrease with increasing plasticity and increasing energy, whether it be applied or stored. The relationship between plasticity and energy is much more clearly illustrated in FIGURE V.13. In this figure it is apparent that there is a tendency for the amount of energy applied or stored to increase with increasing plasticity. This trend is also illustrated in FIGURE V.11, although it is not quite so apparent.

5.3 Summary

The results of the isotropic consolidation and undrained compression tests were presented in this chapter. It was generally shown that very little difference in the test results existed between the natural silt and the kaolin modification, but that the difference was considerable between the montmorillonite modification. Both the natural silt and the kaolin modification were shown to exhibit a dilatant nature. The generally accepted fact that a decrease in the angle of shearing resistance occurs with increasing plasticity was again pointed out for these soils. It was also shown that over-consolidation had very little effect on the effective angle of shearing resistance for the soils and test conditions employed in this work. Results were presented that tend to confirm the known linear relationships on a semi-logarithmic plot between void ratio, and confining pressure and deviator stress. Although this relationship need not exist for over-consolidated samples, it was shown to be as equally applicable for the soils and test conditions employed. It was shown that a linear

relationship appears to exist between applied energy and deviator stress, and that increasing energy application is required with increasing plasticity, in order to achieve the same strengths. The linear relationship does not appear to be valid for over-consolidated samples, with the departure apparently increasing with increasing plasticity. It was shown that a tendency appears to exist for the angle of shearing resistance to decrease with increasing energy application or storage.

From the results presented it generally seems apparent that, unless montmorillonite is present, no significant contribution will be made to the characteristics of a soil by the presence of small amounts of the clay mineral kaolinite. This is also likely to be the case, though to a lesser degree, for the clay mineral illite. However, more work is required on soils modified by the various clay minerals, before anymore generalities can be put forth.

CHAPTER VI

DISCUSSION OF RESULTS

8.1 Introduction

As has been noted previously, this research was undertaken to provide additional information concerning any relationships between soil strength characteristics and energy as applied in an isotropic consolidation process. The first portion of this discussion will be confined to a general discussion of the strength parameters obtained for the three soil types. The second portion of the discussion will be concerned with the relationships that exist between some of the strength parameters obtained and energy. Finally, the significance of the results of this research will be discussed with regard to the work performed by Brooker (1967) on strain energy, and the relevance of this work to Bjerrum's hypothesis.

6.2 Strength Parameters

The effects or influence of clay minerals on the properties of a soil may be readily studied from typical plots such as those presented in the preceding chapter. It has been shown that the addition of a small amount of kaolin to the original silt has altered the properties relatively little, whereas the addition of the same amount of montmorillonite has altered the properties much more significantly.

These results could have been predicted, at least qualitatively, from the index properties obtained for the soil modifications. However, in order to obtain more quantitative information on the effects of clay minerals on certain properties of the soils, tests such as those performed for this thesis are valuable.

It has been shown that the curves of deviator stress, effective stress ratio, pore pressure, and pore pressure parameter, $\bar{A}$, versus strain for the natural silt and kaolin modification are similar. It has also been pointed out that both soils exhibit a strong tendency to dilate when sheared. This characteristic was first noted during the sample preparation stage, where both soil types were noted to respond very readily to the simple shaking test. The curves of pore pressure and pore pressure parameter, $\bar{A}$, versus strain also tend to confirm this characteristic.

For the original soil and kaolin modification it was noted that the deviator stress showed some tendency to peak at low strains (1 to 2 percent), but with additional strain this tendency disappeared and the deviator stress increased substantially. The peak value occurred at strains in the order of 16 to 20 percent. It was also apparent from the pore pressure curves that the pore pressure was decreasing as the deviator stress increased. It is felt that this behaviour can be attributed to the strong tendency for dilation that exists in these two soils. It is considered that even though expansion is prevented for all practical purposes in the undrained test, the tendency for expansion exists at the points of soil particle contact.

This tendency for expansion reduces the overall positive pore pressure measured by an amount equal to the tension transmitted to the pore water. Since a reduction in the pore pressure results in an increase in the effective stress between particles, the deviator stress increases. So long as this tendency for dilation exists, an increase in deviator stress will result.

Samples prepared from the montmorillonite modified soil showed no tendency for dilation under shear. This may be explained on the basis that montmorillonite is a very surface-active clay mineral and has the ability to build up very thick layers of adsorbed water. These water hulls overlap, and as a result, effectively form a physical-chemical bond between the clay particles which augment other interparticle attractions. Although these bonds may be weak, becoming increasingly so with increasing water content of the sample, it is felt that they possess sufficient strength to take up any tension that would result from a tendency for the soil to dilate, and hence, no dilation would occur. This explanation is based on the physically observable phenomenon that minute expansion results in substantial pressure reductions in pressurized systems. This being the case, then only a small tendency to dilate would be required to reduce the pore water pressure substantially. However, employing the same reasoning, only a small resisting force would be required to prevent dilation.

The general concept that a decrease in the effective angle of shearing resistance occurs with increasing plasticity was substantiated by these test results. It was found that the greatest decrease resulted for the soil modified by the highly surface-active clay mineral

montmorillonite. For cohesionless soils the void ratio and degree of particle interference or interlock are known to depend upon particle size, shape and gradation. These factors are also known to affect the angle of shearing resistance developed. Generally, an increase in the angle of shearing resistance occurs with decreasing void ratio and increasing particle interference. It would appear that the same should hold true for highly cohesive soils. However, the interference effects are almost absent due to the presence of the large viscous water hulls surrounding the clay particles. These water hulls, although they tend to bind the particles together, cannot appreciably withstand shearing stresses, and thus, contribute little to interparticle interference. Since these water hulls become progressively larger for the more active clay minerals it seems logical that a decrease occurs in the effective angle of shearing resistance with increasing plasticity. This is not the sole factor responsible for the decrease as particle size and shape are also contributing factors.

Another finding of interest in this series of tests was that the effective angle of shearing resistance was not affected by over-consolidation. This conclusion is drawn from the fact that the points for the over-consolidated samples tend to fall on the same envelope as those for the normally consolidated samples. At first hand these results appear to be contradictory to those generally obtained for undisturbed over-consolidated field specimens, where the strength envelopes are characterized by a cohesion intercept and a reduced angle of shearing resistance as illustrated in FIGURE VI.1.

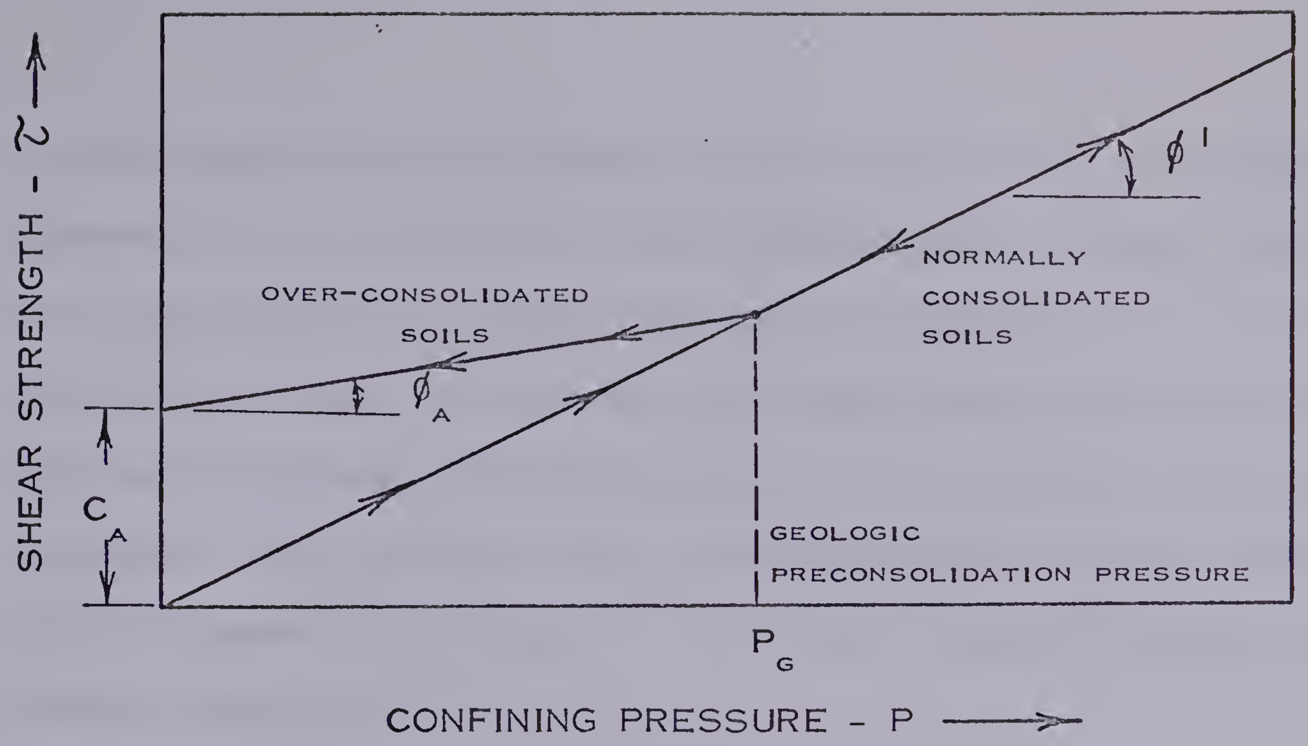

FIGURE VI.1 TYPICAL RELATIONSHIPS BETWEEN SHEAR STRENGTH AND CONFINING PRESSURE [AFTER TAYLOR, 1948]

However, this is not really the case as the over-consolidated laboratory prepared specimens have not been subjected to the same time element as those taken from the field. In the course of geologic time the soil in the field has developed an interparticle bonding which is responsible for the apparent cohesion. The time element in the laboratory is far too small to allow the development of interparticle bonds of such a nature, and hence, the strength is attributed solely to interparticle friction. In the case of pure silts, interparticle bonding is not a factor in any case.

Taylor (1948) showed that a linear relationship existed on a semi-logarithmic plot of void ratio versus confining pressure or deviator stress for normally consolidated samples. This linear relationship, however, was also found to hold true for the over-consolidated samples within the test conditions employed here. In light of the pre-

ceding discussion on the effects of over-consolidation on the strength parameter ϕ' , this seems to be an understandable finding. Since over-consolidation apparently had little or no effect on ϕ' , it seems that a linear relationship is as appropriate in this case for the over-consolidated specimens as it is for the normally consolidated specimens. This fact was found to be advantageous in that a method of least squares was employed to fit straight lines to the data and thereby eliminate bias in the results.

6.3 Energy-Strength Relationships

For the soils tested, the conditions of normal consolidation and within the limits of pressure employed, it was found that a linear relationship existed between the deviator stress at failure and the energy applied. The energy applied is the energy expended in consolidating the specimen to some final void ratio, and is computed as the area under the confining pressure - void ratio curve bounded by the particular confining pressure and void ratio attained.

Considering the results of this research it seems that perhaps the linear relationships obtained between deviator stress and applied energy for the normally consolidated samples could have been predicted on the basis of already known facts. For normally consolidated homogeneous soils a direct proportionality exists between shear strength and depth of over-burden or over-burden pressure (confining pressure). It may be argued that perhaps a linear relationship could have been predicted since the deviator stress at failure is a measure of the shear strength of a soil, and the energy applied in consolidating the sample

to some void ratio is a function of the confining pressure. However, it should be pointed out that the energy applied is also a function of the distortion or volume change of the soil sample as well as the confining pressure, and is in fact the product of the two. This would tend to complicate any prediction of the relationship between strength and energy as uniaxial or triaxial compression bears a logarithmic relationship to pressure and hence energy. Regardless of whether the results could have been predicted or not, the data of this thesis are quantitative evidence that for the soils tested, and within the range of confining pressures employed, a linear relationship exists between strength and energy applied in the consolidation process for normally consolidated soils. For a perfectly elastic soil, the curve for the over-consolidated specimens, which represents the energy liberated, would coincide with that of the normally consolidated specimens, which represents the energy applied. For the soils tested, it is evident that neither soil is ideally elastic and that the departure from this state becomes increasingly pronounced as active clay minerals are introduced into the original soil system. That is to say, the plasticity, as indicated by the plasticity index, is increased by the addition of the clay minerals kaolinite and montmorillonite. Since the clay mineral montmorillonite is much more active than the clay mineral kaolinite, the addition of an equal portion of montmorillonite to a soil system results in a much greater change in the original soil properties. This fact is reflected not only in the shape of the rebound curve on the deviator stress-energy plot, but in the distance between the loading and rebound branch which represents the energy dissipated

within the soil system. The increased energy loss is likely due to energy lost in deforming the viscous water hulls surrounding the clay particles and reorientation of the clay particles. Some energy may be retained by interparticle bounding which is likely to increase with increasing activity of the clay mineral. However, since the consolidation time in the laboratory is of such a short duration, it is unlikely that any bonding other than the weak physical-chemical bonding occurs.

The results of this thesis research point out several other features. It is evident that for a given strength, the energy absorption capacity increases as the plasticity increases. This tendency becomes very pronounced for soils containing the highly active clay mineral montmorillonite. This finding can be explained on the basis of the affinity of this clay mineral for water. If the three soils tested were compared on the basis of having the same relative consistency $[I_c = (W_L - W_n) / I_p]$ it would be found that the montmorillonite would have the highest moisture content and the unmodified silt the lowest moisture content. If this were the case then a great deal more energy would be required in consolidating the montmorillonite samples to a common void ratio and strength than would be required for the natural soil. In other words, for the same relative consistency the montmorillonitic samples exist at much higher void ratios and more energy is thus required in consolidating them.

The results also indicate that there is some relationship existing between plasticity and the effective angle of shearing resistance, and energy both stored and applied. Although the results are too limited to draw any definite conclusions trends clearly exist.

It is apparent that the total amount of applied and stored energy increases with increasing plasticity index. This relationship has been previously mentioned in discussing energy-strength relationships. However, it becomes much more evident when the plasticity index is plotted versus total applied and stored energy. There is also a definite trend with regard to the variation of the effective angle of shearing resistance with energy. The results indicate that as the energy application and storage increase, the effective angle of shearing resistance decreases, and that an inter-relationship exists between plasticity, the angle of shearing resistance and energy.

6.4 Significance of Results

Although the results of this thesis research apply only to the soils and test conditions utilized, they tend to show a definite relationship between energy application and strength. They are by no means conclusive, but appear to be important from the point of view that they show that strength appears to be directly proportional to energy applied for the normally consolidated soils and that for the over-consolidated soils energy retention appears to be some function of the clay mineralogy of the soil.

The results of this thesis indicate that the effective angle of shearing resistance, for the soils and test conditions utilized, bears a definite relationship to stored energy. Brooker (1967) found the same to be true for the soils he tested. The results are interesting from the point of view of Bjerrum's hypothesis. It is known that the development of diagenetic bonds is a function of mineralogy as well as

the magnitude and duration of pressure, and is related to the locked-in strain energy. If a clearer understanding can be obtained of the relationships that exist between diagenetic bonding, stored energy, and the effective angle of shearing resistance, then perhaps a better understanding of the behaviour of over-consolidated clays and the phenomena of long-term slope failures may be achieved. It is felt that the present study together with Brooker's findings has lent some support to the concept that the strength of highly over-consolidated clays is related to the energy stored by these clays.

6.6 Future Research

It would be of interest to continue this type of testing utilizing much higher confining pressures. Cell pressures in the order of 1500 to 2000 pounds per square inch would be more nearly equivalent to the estimated preconsolidation or over-burden pressures that have existed on some of the soils in this area. It would be of special interest to see whether or not the linear relationship between strength and energy continues to exist for these high pressures and what effects these pressures would have on the over-consolidated samples and their characteristics. Very much higher over-consolidation ratios could be achieved and it may be found that under these conditions of compaction, interparticle bonding may begin to take place on a more significant scale. If this were found to be the case it would have a great deal of practical significance as Bjerrum (1967) bases his progressive failure hypothesis on the breakdown of diagenetic bonds.

In order to isolate some of the factors which influence strength and energy absorption characteristics, a pilot research project such as this must initially be confined to laboratory prepared, simplified soil systems. Future research may consider other factors or combinations of factors. However, in order to relate the findings of such research to actual field conditions, work will ultimately have to be performed on undisturbed field specimens, under simulated field conditions.

6.6 Summary

This chapter attempted to analyze and explain through basic soil mechanics concepts the results presented in Chapter V.

A discussion of the effects of mineralogy on the strength parameters obtained was presented. In addition the phenomenon of soil dilation under shear was related to mineralogy. It was shown, as in the past, that the effective angle of shearing resistance decreased with increasing plasticity. Because of the relationship of plasticity to mineralogy a discussion was presented explaining the effects of mineralogy on the effective angle of shearing resistance. Over-consolidation was shown to have no effect on the effective angle of shearing resistance and a discussion of the reasons why this should be so, in light of accepted findings, was presented. It was demonstrated that a linear relationship on a semi-logarithmic basis existed between void ratio, and confining pressure and deviator stress at failure for normally consolidated soils. The same was shown to be true for the

over-consolidated soils and a brief discussion of why this should be the case was presented.

Applied energy was found to be linearly porportional to deviator stress for normally consolidated samples, and was found to depart from linearity for the over-consolidated samples. The departure from linearity for the over-consolidated soils was noted to increase with increasing plasticity. A discussion relating plasticity to applied and stored energy was then presented. It was also shown that a relationship exists between the effective angle of shearing resistance and applied and stored energy.

The significance of the test results was discussed with regard to work performed by Brooker and the relationship of the test results to Bjerrum's hypothesis was presented. Finally, comments were made regarding additional research in this direction.

CHAPTER VII

CONCLUSIONS

The research presented in this thesis consisted of a testing program on a pure silt from the Alaska Highway, and kaolin and montmorillonite modifications of this silt. The consolidated - undrained triaxial test with pore pressure measurements was employed to test remoulded normally consolidated and over-consolidated samples. Arising from this work, for the test conditions employed and materials tested, the following conclusions are drawn:

1. The effects of equal amounts of kaolin and montmorillonite on the properties of a soil are very different.

2. For the amount of kaolin and montmorillonite used, the kaolin, having a low surface activity, has almost a negligible effect on the original soil properties, whereas, montmorillonite has a marked effect on them. This point is illustrated by plots of deviator stress, effective stress ratio, pore pressure, and pore pressure parameter, $\bar{A}$, versus axial compressive strain. In all cases, the original soil and the kaolin modification exhibit similar curves. The montmorillonite modified soil, however, exhibits curves which are considerably different in both shape and magnitude.

3. A linear relationship exists between strength and applied energy for the normally consolidated soils.

4. For the over-consolidated soils the relationship between energy and strength is non-linear and becomes increasingly so with increasing plasticity.

5. Energy liberation and retention for over-consolidated soils is a function of the clay mineralogy of the soil. This point is illustrated by the fact that increasingly greater amounts of energy are required to attain comparable strengths as the plasticity increases. This is very apparent for the montmorillonite modification.

6. Although the results are limited, there is a clearly defined tendency for a decrease in the effective angle of shearing resistance with increasing applied and stored energy.

7. There is a definite inter-relationship between plasticity, as indicated by the plasticity index, the effective angle of shearing resistance, and stored and applied energy.

8. This research indicates that unless the clay mineral montmorillonite is present in a soil system, there will be no appreciable problems with regard to the determination of the strength and volume change properties of the soil. The presence of even small portions of montmorillonite tends to alter the properties of the soil mass drastically.

9. The techniques employed with regard to pore pressure reaction tests did not result in high pore pressure reactions, however, this does not preclude the measurements being satisfactorily accurate.

10. The technique employed for sample preparation yielded high quality specimens.

CHAPTER VIII

RECOMMENDATIONS

The following recommendations arising from this thesis are offered.

1. Future research should include soils modified with varying amounts of the same clay mineral in order to establish the effects of varying plasticity on such properties as strength and energy absorption characteristics.

2. It would be desireable to carry out further research at confining pressures in the order of 1500 to 2000 pounds per square inch in order to more closely simulate field consolidation pressures.

3. In order to extend this research, it is recommended that use be made of direct or ring shear apparatus. By using these devices energy is applied under anisotropic loading conditions and both peak and residual strength parameters can be obtained. It may be argued by some that anisotropic loading conditions are more representative of field conditions than are isotropic loading conditions.

4. Ultimately it will be desireable to use undisturbed field specimens, which generally are a complex aggregation of various minerals and may have complex stress histories. In this way perhaps some insight can be obtained into the behaviour of soils in the field, and

what effects are contributed to strength and energy absorption by compression for geologic time intervals.

5. In future undrained triaxial tests with pore pressure measurements, it is desireable to use back pressures of 30 pounds per square inch and make full use of internal and external drainage-aids, provided that these conditions can be achieved, and their implimentation does not adversely affect the condition of the sample.

6. The technique employed for the preparation of samples for this research should be employed when dealing with very dilatant soils. It is felt that this procedure contributed the least to sample disturbance and it is apparent from the results that it yields samples with fairly consistent properties.

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A P P E N D I X " A "

TRIMMING AND MOUNTING THE TRIAXIAL TEST SPECIMEN

APPENDIX A

TRIMMING AND MOUNTING THE TRIAXIAL TEST SPECIMEN

The following is the sequence of steps used in trimming and mounting the triaxial specimens:

1. The consolidation tube with sample was removed from the loading frame, the end porous stone was removed and all external moisture on the tube was wiped off.

2. Calipers were used to mark off several lengths of 80.0 mm from one end of the tube. These marks served as a guide to trimming the specimens to the desired length (80.0 mm) as extrusion and trimming proceded from one end only.

3. After the specimen had been trimmed to the desired length and the trimmings had been retained for an initial moisture content, all excess soil and moisture was cleaned from the tube.

4. The tube plus specimen was then immediately weighed to the nearest hundredth of a gram. Since the tares of all tubes had been pre-determined, the initial weight of the triaxial specimen could be easily calculated.

5. The sample dimensions were ascertained by measuring the tube diameters at top and bottom. Two measurements were made in each position to the nearest 0.01 mm and averaged. The initial cross-sectional area was taken as the average of the areas at the top and bottom of the tube.

6. The pore pressure lines in the pedestal and base of the triaxial cell were flushed with distilled water. The lines were left full by connecting a twin-burette volume change indicator to one line, and closing off the other with a no-volume-change slide valve.

7. A saturated porous ceramic stone covered by a filter paper was placed in contact with the pedestal. A piece of rubber tubing was placed over the pedestal in such a manner that the tubing covered the edge of the stone. The same procedure was followed on the top loading head.

8. The specimen was extruded directly from the tube onto the pedestal and wrapped in a saturated slotted filter paper which was allowed to come in contact with the bottom stone.

9. The top loading cap was then centered on the specimen and the entire assemblage was encased in one rubber membrane with the aid of a membrane stretcher. The outside of the membrane had been greased prior to placement with a thin layer of silicone grease.

10. The membrane was fixed to the pedestal and loading cap by three rubber "O" rings in each location.

11. The triaxial cell was secured in position and the chamber filled with distilled water. An oil seal of approximately 1/2 inch was used at the top of the cell in order to maintain pressure for prolonged periods.

12. The loading piston was placed in position and clamped.

13. Free air in the cell was removed by forcing it out the top valve. The pore pressure lines were flushed once again with distilled water to remove any air present between membrane and sample.

14. With the volume change indicator line open, the cell pressure and back pressure were simultaneously built up to 1 Kg/sq cm by means of the screw controls. The constant pressure cell for back pressuring was tied into the volume change system and the volume change indicator valve shut off.

15. The cell pressure was built up to the desired confining pressure and another constant pressure cell brought into the cell pressure system.

16. The volume change indicator valve was opened and a stopclock started. In this manner plots of volume change versus log-time were obtained.

17. Consolidation was carried out to at least theoretical 100 percent consolidation at which time the volume change indicator valve was closed off.

18. After consolidation was complete a pore pressure transducer was coupled to the slide valve, the valve was opened, and distilled water was flushed through the assembly to remove entrapped air.

19. The triaxial cell was then mounted on the loading press with a load cell in contact with the loading piston.

A P P E N D I X "B"

SUMMARY OF DATA SHEETS

TABLE B.1

SUMMARY OF TEST DATA - ALASKA HIGHWAY SILT

Sample No.	σ_3 (Kg/cm^2)	O.C.R.	Moisture Content-% Initial	Moisture Content-% Final	Initial* Degree of Saturation %	Void Ratio Initial (e_i)	Void Ratio Final (e_f)	Max. σ_1'/σ_3'	$(\sigma_1-\sigma_3)$** (Kg/cm^2)	Strain %	Pore Pressure Reaction %
2	2	1.00	22.63	22.05	100.04	0.604	0.589	3.848	2.187	6.0	87
3	4	1.00	23.15	20.93	102.41	0.618	0.559	2.989	5.634	11.0	36/44
4	6	1.00	22.68	20.68	99.43	0.606	0.552	3.637	7.365	9.5	66
5	4	1.00	22.58	21.07	98.10	0.614	0.563	3.801	6.191	10.0	38
7	8	1.00	23.25	20.40	102.10	0.621	0.545	3.679	11.764	11.0	66
12	3	1.00	23.32	21.74	101.02	0.623	0.580	4.150	3.257	10.0	69
8	6	1.33	23.41	20.56	106.20	0.625	0.549	4.217	8.819	8.0	33
9	4	2.00	22.76	20.62	100.40	0.605	0.551	3.793	8.676	10.0	60
10	2	4.00	23.03	20.95	98.30	0.626	0.559	3.763	5.998	9.5	68
11	3	2.67	23.56	21.18	99.67	0.631	0.566	4.333	5.757	8.0	61
13	3	2.67	23.18	21.34	99.04	0.625	0.570	3.453	6.279	10.0	87

* Final degree of saturation assumed to be 100%, as samples extremely sensitive to handing and dilated very readily.

** Deviator stress corresponding to Max σ_1'/σ_3' .

TABLE B.II

SUMMARY OF TEST DATA - KAOLIN MODIFIED ALASKA HIGHWAY SILT

Sample No.	σ_3 (Kg/cm^2)	O.C.R.	Moisture Initial	Content-% Final	Initial* Degree of Saturation %	Void Initial (e_i)	Ratio Final (e_f)	Max. σ_1'/σ_3'	$(\sigma_1 - \sigma_3)$*** (Kg/cm^2)	Strain %	Pore Pressure Reaction %
1	2	1.00	23.08	21.43	92.54	0.686	0.589	3.505	2.761	7.5	91
2	4	1.00	23.03	21.05	95.60	0.662	0.579	4.394	4.114	7.5	61
3	6	1.00	23.39	20.67	93.56	0.688	0.568	3.516	5.675	9.0	61
5	8	1.00	22.71	19.42	94.49**	0.661	0.550	3.545	16.361	15.0	64
9	6	1.00	23.74	20.66	94.81	0.688	0.568	3.375	5.211	9.0	86
10	2	1.00	23.24	21.96	91.67	0.697	0.604	4.029	2.075	8.0	64
6	6	1.33	23.19	20.21	94.37	0.676	0.556	3.469	9.319	12.0	40
7	4	2.00	23.60	20.49	93.47	0.694	0.563	3.432	6.015	10.0	80
8	2	4.00	23.36	20.84	95.12	0.675	0.573	3.479	4.221	7.0	76

* Final degree of saturation assumed to be 100%, as samples sensitive to handling and dilated very readily.

** Final degree of saturation calculated to be 97.04%.

*** Deviator stress corresponding to Max σ_1'/σ_3' .

TABLE B.III

SUMMARY OF TEST DATA - MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT

Sample No.	σ_3 (Kg/cm²)	O.C.R.	Moisture Initial	Content-% Final	Initial* Degree of Saturation %	Void Initial (e_i)	Ratio Final (e_f)	Max. σ_1'/σ_3'	$(\sigma_1 - \sigma_3)$**** (Kg/cm²)	Strain %	Pore Pressure Reaction %
1	2	1.00	32.27	28.96	95.95	0.921	0.791	2.846	1.034	7.5	73
2	4	1.00	32.21	26.24	96.64	0.910	0.716	2.700	1.841	9.0	76
3	6	1.00	31.93	24.47	91.22***	0.956	0.668	2.771	2.930	8.5	46
4**	8	1.00	32.09	23.63	96.02	0.912	0.645	-	-	-	68
5	8	1.00	31.96	23.23	96.37***	0.905	0.634	1.750	3.898	10.0	35
6	6	1.33	33.82	23.62	97.60	0.946	0.645	2.958	3.428	10.0	27/31
7	4	2.00	32.12	24.03	96.89	0.905	0.656	3.014	2.920	12.0	26
8	2	4.00	32.82	24.02	97.48	0.919	0.656	2.730	2.675	5.0	55

* Final degree of saturation assumed to be 100%.

** Leak developed around pore pressure transducer - test terminated at 3% strain. (Pore pressure readings questionable.) Results of this test were not used in final analysis.

*** Final degree of saturation calculated to be 100.72% and 100.26% for samples 3 and 5 respectively.

**** Deviator stress corresponding to Max σ_1'/σ_3' .

A P P E N D I X " C "

TYPICAL DATA SHEETS AND
SAMPLE CALCULATIONS FOR THE
CONSOLIDATED UNDRAINED TRIAXIAL TEST

Sheet 1

C.1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth Sample # 4

Engineer L.A. BALANKO

Technician

Data at Failure (USING $\bar{\sigma}_1/\bar{\sigma}_3$ AS CRITERION)

$(\sigma_1 - \sigma_3)$ = 7.365 KG/CM² σ_1 = 13.365 KG/CM²

P = 3.207 KG/CM²

$\bar{\sigma}_1$ = 10.158 KG/CM²

$\bar{\sigma}_3$ = 2.793 KG/CM²

Strain 9.5 %

Date of Test FEB 9/68

Test Lateral Pressure σ_3 = 7-1 = 6 KG/CM²

Back Pressure 1 KG/CM²

Remarks RATE OF STRAIN ~2.25%/HR.
PORE PRESSURE -1 μ"/" = 8.8719×10⁻⁴ KG/CM
LOAD CELL -1 μ"/" = 0.033267 KG.

Area Correction Factor 1.024

Time min FEB 10/68	Strain Dial Div.	A_c cm²	LOAD CELL ~~No. of Stress Dial Div.~~ RDG.	LOAD CELL ~~Proving Ring Const~~ δ, kg/Div.	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\bar{\sigma}_1/\bar{\sigma}_3$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$
1000ᴴ	0	9.096	10025	0	0	21290	6.000	6.000	1.000	0.00	0
	16	9.114	10055	30	0.112	322 .028	6.083	5.972	1.019	0.2	.250
	32	9.133	10072	47	0.175	365 .067	6.108	5.933	1.029	0.4	.383
	48	9.151	10517	492	1.831	22060 .683	7.148	5.317	1.344	0.6	.373
	64	9.169	10808	783	2.909	832 1.368	6.923	4.632	1.495	0.8	.470
1027ᴴ	80	9.188	10962	937	3.474	23534 1.991	7.483	4.009	1.867	1.0	.573
	96	9.206	11065	1040	3.848	24090 2.484	7.364	3.516	2.094	1.2	.646
	112	9.225	11120	1095	4.043	522 2.867	7.176	3.133	2.290	1.4	.709
	128	9.244	11160	1135	4.183	780 3.096	7.087	2.904	2.440	1.6	.740
	144	9.263	11180	1155	4.247	25000 3.291	6.956	2.709	2.568	1.8	.775
1054ᴴ	160	9.282	11200	1175	4.312	136 3.412	6.900	2.588	2.666	2.0	.791
	176	9.301	11220	1195	4.377	268 3.529	6.848	2.471	2.771	2.2	.806
	192	9.320	11238	1213	4.434	345 3.598	6.836	2.402	2.846	2.4	.811
	208	9.339	11255	1230	4.487	422 3.666	6.821	2.334	2.922	2.6	.817
	224	9.358	11278	1253	4.561	474 3.712	6.849	2.288	2.993	2.8	.814
1126ᴴ	240	9.377	11295	1270	4.614	491 3.730	6.884	2.270	3.033	3.0	.808
	280	9.426	11345	1320	4.770	563 3.791	6.979	2.209	3.159	3.5	.795
	320	9.475	11400	1375	4.993	580 3.806	7.137	2.194	3.253	4.0	.770
	360	9.525	11460	1435	5.132	562 3.790	7.342	2.210	3.322	4.5	.738
	400	9.575	11525	1500	5.337	545 3.775	7.562	2.225	3.399	5.0	.707
	440	9.625	11590	1565	5.539	492 3.728	7.811	2.272	3.438	5.5	.673
	480	9.677	11660	1635	5.756	450 3.691	8.065	2.309	3.493	6.0	.641
	520	9.728	11735	1710	5.988	390 3.637	8.351	2.363	3.534	6.5	.607
1306ᴴ	560	9.781	11808	1783	6.210	318 3.574	8.636	2.426	3.560	7.0	.576
	600	9.834	11888	1863	6.453	245 3.509	8.944	2.491	3.590	7.5	.544
1332ᴴ	640	9.887	11960	1925	6.632	168 3.441	9.191	2.559	3.592	8.0	.519
	680	9.941	12035	2010	6.888	085 3.367	9.521	2.633	3.616	8.5	.489
	720	9.986	12113	2088	7.116	24992 3.284	9.832	2.716	3.620	9.0	.461
	760	10.061	12198	2173	7.365	905 3.207	10.158	2.793	3.637	9.5	.435

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth Sample #4

Area Correction Factor 1.024

Time min	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm2 P_P		Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$
1425[H]	800	10,107	12270	2245	7.567	24810	3.123	10.444	2.877	3.630	10.0	0.413
	880	10.220	12420	2395	7.983	610	2.945	11.038	3.055	3.613	11.0	0.369
	960	10.336	12520	2495	8.223	407	2.765	11.458	3.235	3.542	12.0	0.336
	1040	10.455	12582	2557	8.331	263	2.638	11.693	3.362	3.478	13.0	0.317
	1120	10.577	12670	2645	8.519	152	2.539	11.980	3.461	3.461	14.0	0.299
	1200	10.701	12680	2655	8.451	152	2.539	11.912	3.461	3.442	15.0	0.300
	1280	10.829	12730	2715	8.541	142	2.530	12.011	3.470	3.461	16.0	0.296
	1360	10.959	12770	2755	8.564	140	2.528	12.036	3.472	3.467	17.0	0.295
				(FAILURE)								

Moisture Content – Initial 22.68%

Final 20.68%

Degree of Saturation – Initial 99.43%

$S = \frac{\gamma_{wet}\, \omega\, G_s}{[\gamma_{wet} - G_s(1+\omega)]}$ Final 100% (ASSUMED)

Void Ratio – Initial 0.606

Final 0.552

Pore Pressure Reaction 66.3%

Sketch of Failure

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth sample #4

Engineer L.A. BALANKO

Technician

Date of Set-up FEB 7/68

Test Lateral Pressure $\sigma_3 = 7-1 = 6 KG/CM^2$

Sample Description ALASKA HIGHWAY SILT - VERY LITTLE DRY STRENGTH - LOW PLASTICITY - MEDIUM GREY BROWN IN COLOR - ANISOTROPICALLY CONSOLIDATED @ $1.5 KG/CM^2$

Initial Weight 148.14 gms
(264.17 - 116.03)

	1.	2.	Aver:	
Length, mm			80	Area Top A_T 909.60 sq.mm
Diam. mm Top	33.96	34.12	34.04	Area Centre A_C sq.mm
Centre				Area Bottom A_B 909.60 sq.mm
Bottom	34.02	34.06	34.04	Average X-Sect Area

Original Volume cc

$= \frac{1}{2}(A_T + 2A_C + A_B)$

$= 909.60$

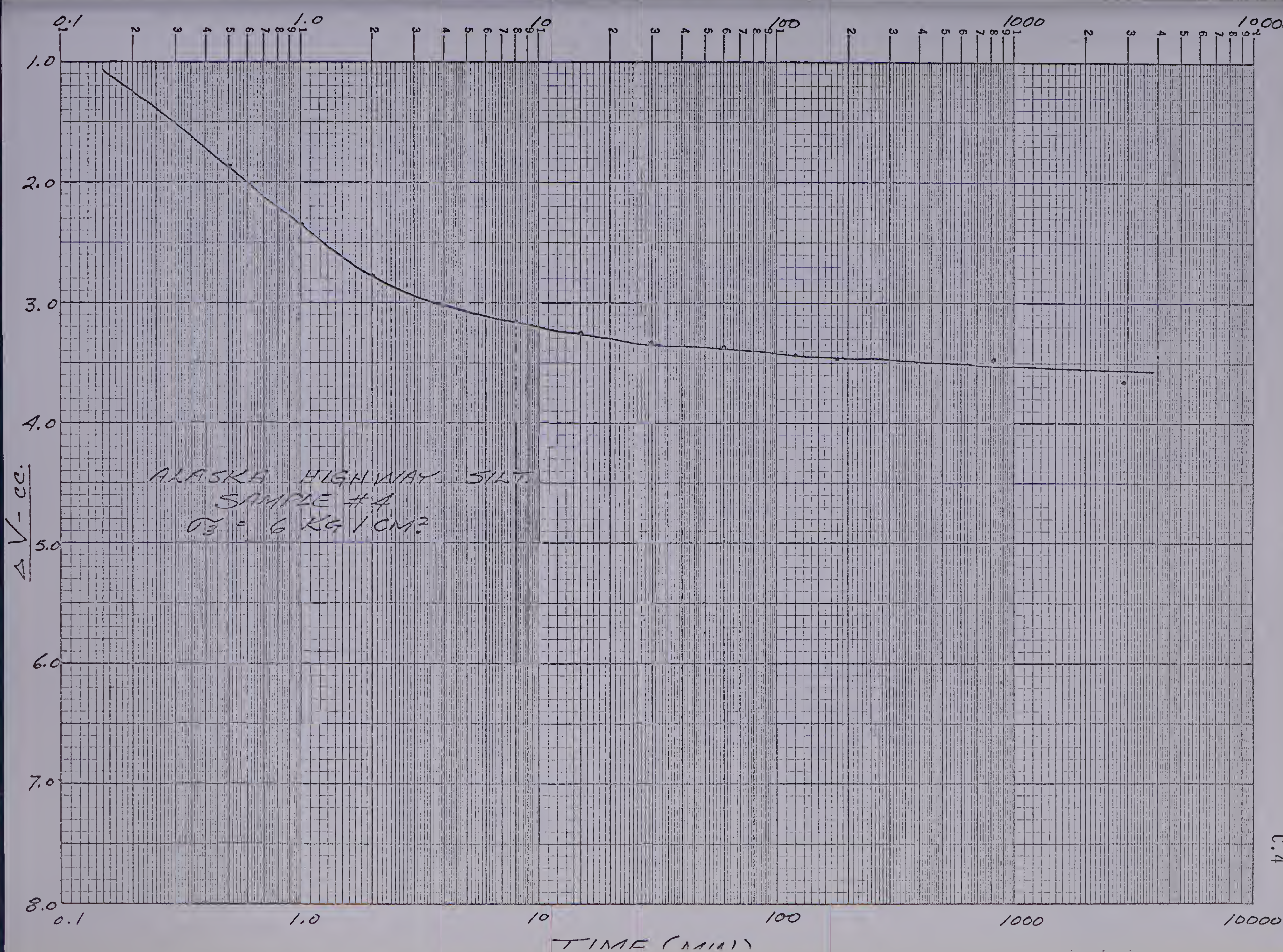
CONSOLIDATION DATA

$\sigma_3 = 6 KG/CM^2$

Date Time	Δt min.	Burette Rdg. c.c. IN L.H.	R.H.	ΔV c.c. (AVG.)
FEB 7/68		L.H.	R.H.	
1950 H	0	2.35	2.19	0.00
	30 S	0.55	4.04	1.87
	1 m	1.01	OUT 3.52	2.36
	2 m	1.44	3.11	2.78
	4 m	1.67	2.88	3.01
	8 m	1.81	2.75	3.14
	15 m	1.90	2.67	3.23
	30 m	1.98	2.59	3.31
2050 H	1 H	2.04	2.52	3.37
2150 H	2 H	2.10	2.45	3.44
2250 H	3 H	2.13	2.42	3.47
FEB 8/68 930 H	13 H 40 m	2.08	2.36	3.47
FEB 9/68 1010 H	38 H 20 m	2.20	2.16	3.62

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.


ALASKA HIGHWAY SILT
SAMPLE #4
σ_3 = 6 KG/CM^2
ΔV - cc.
TIME (MIN)
0.1
1.0
10
100
1000
10000
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No #4
Sample Desc. SILT
Kg/sq. cm on cell 7-1 = 6

$1\mu''/_{11} = 8.8719 \times 10^{-4}$ Kg/cm^2

"LOAD" 6 Kg/cm^2 → 7 Kg/cm^2		"UNLOAD" 7 Kg/cm^2 → 6 Kg/cm^2	
t min	Pp Kg/cm^2	t min	Pp Kg/cm^2
0	21558	0	22305
0.1	22230	0.1	21765
0.25	—	0.25	21715
0.50	22285	0.50	21660
1.0	22295	1.0	21622
2.	22300	2.	21603
3	22302	3	21600
4	22302	4	21600
5	22300	5	21600
8	22302	8	21600
10	22305	10	21603

Pp = 66.3%

	Final Moisture Content	INITIAL M.C.
Wet Wt. plus tare	242.19	104.68
Dry Wt. plus tare	217.53	98.29
Wt. of water	24.66	6.39
Wt. and No. of tare	L1 - 98.30	LB1 - 70.11
Wt. of dry soil	119.23	28.18
Final M.C.	20.68%	22.68%

Final Volume

Wt. Hg + Tare ______

Tare ______

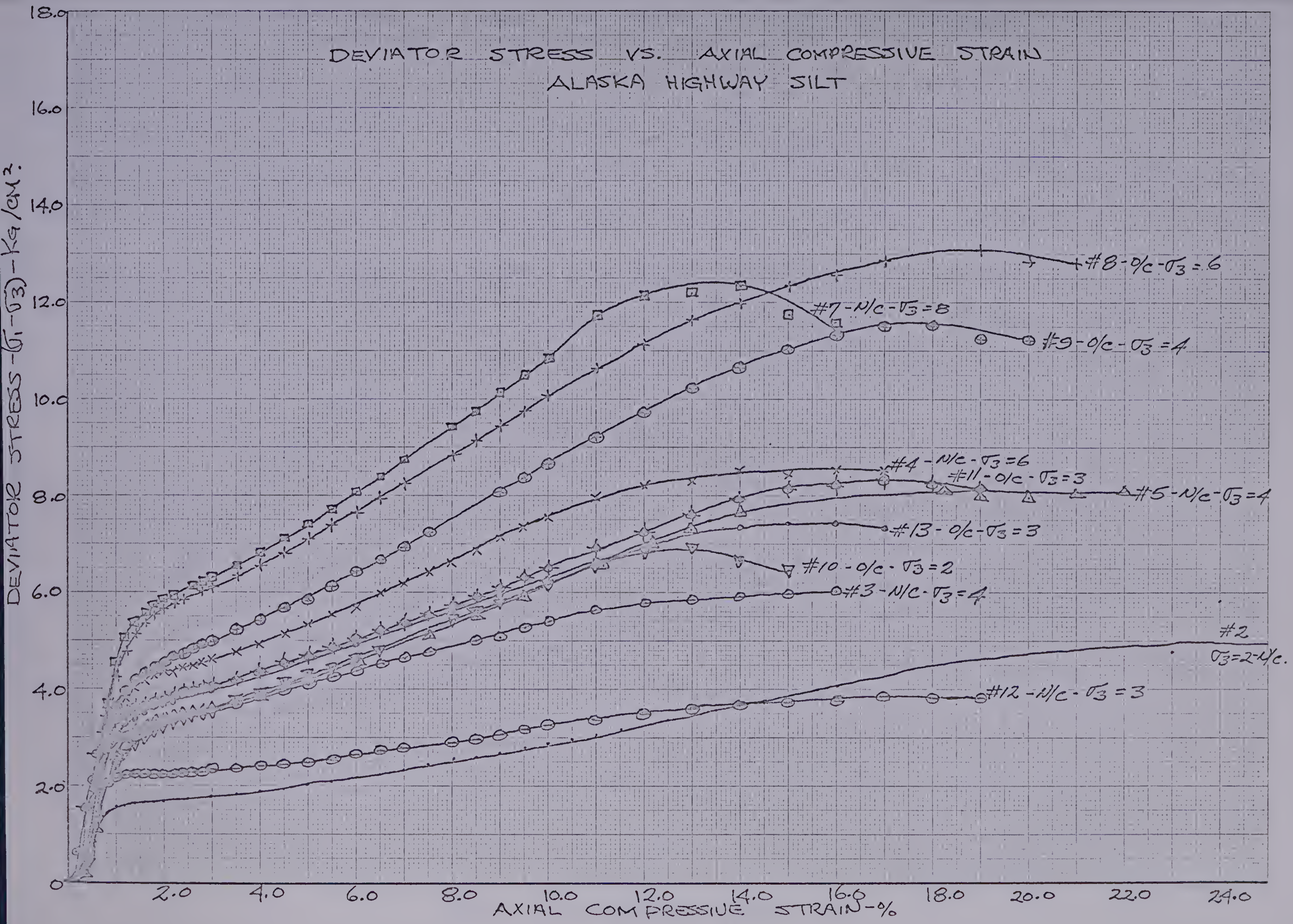
Wt. Hg ______

Temp. ______

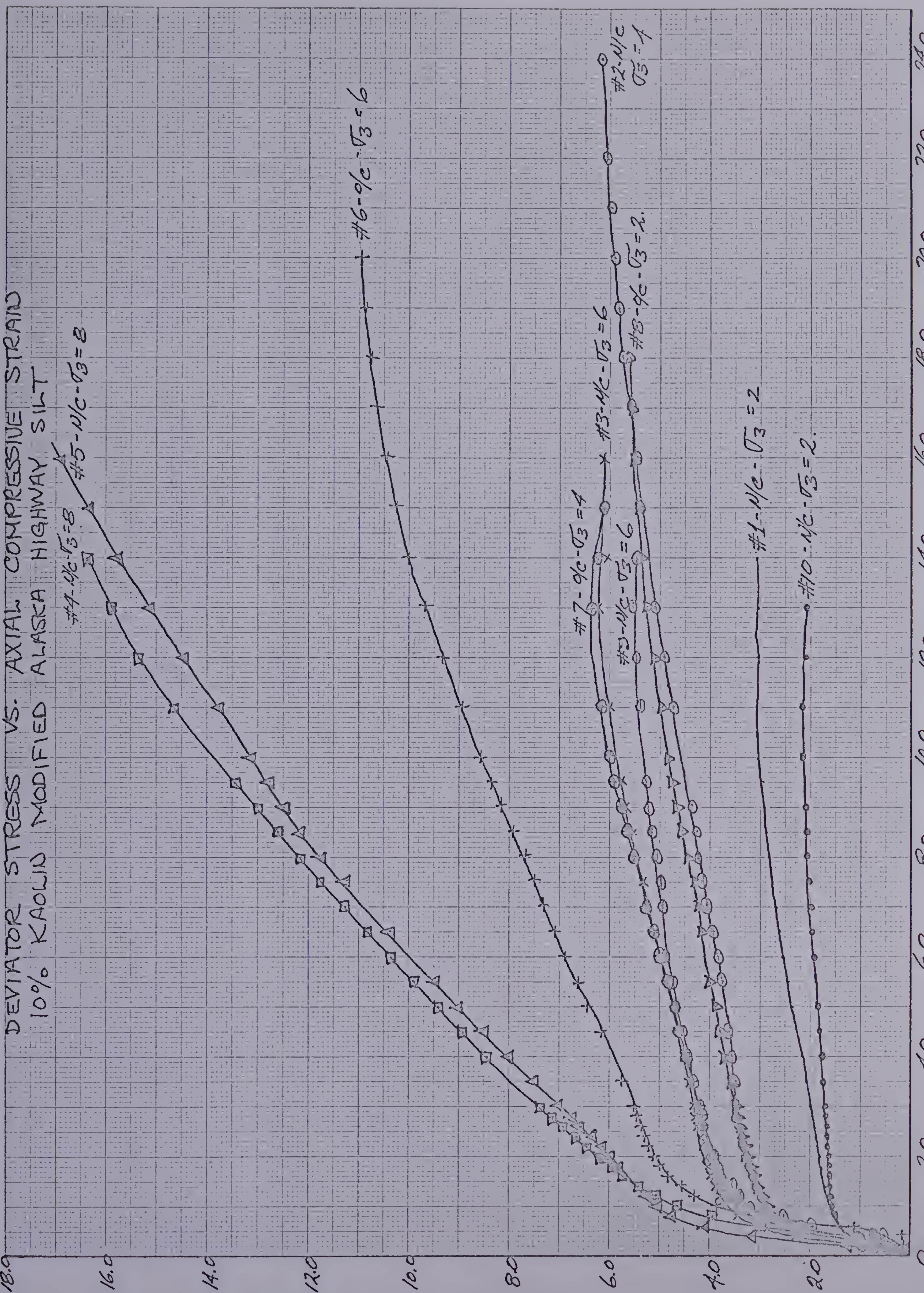
Vol. Hg ______

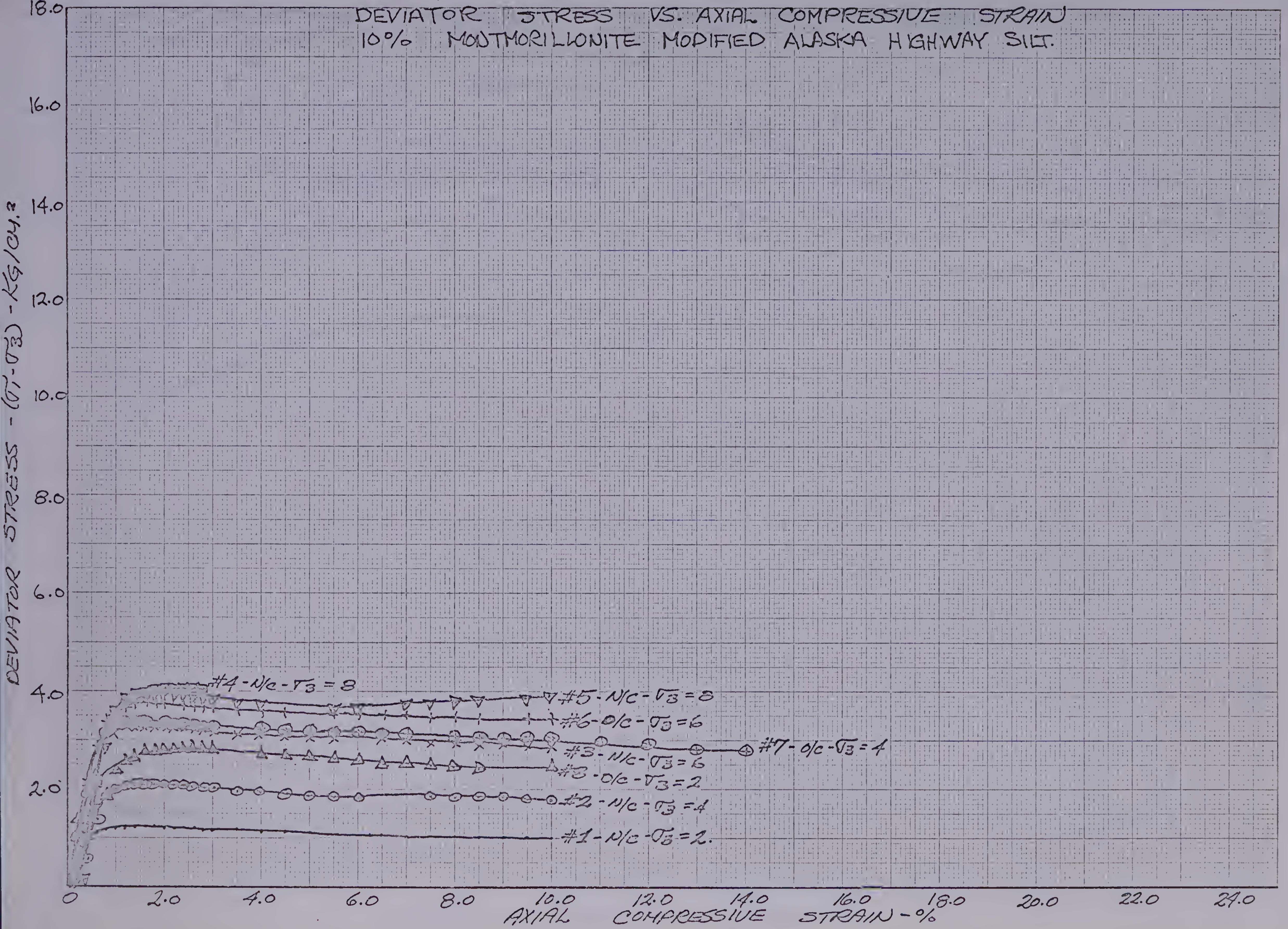
A P P E N D I X " D "

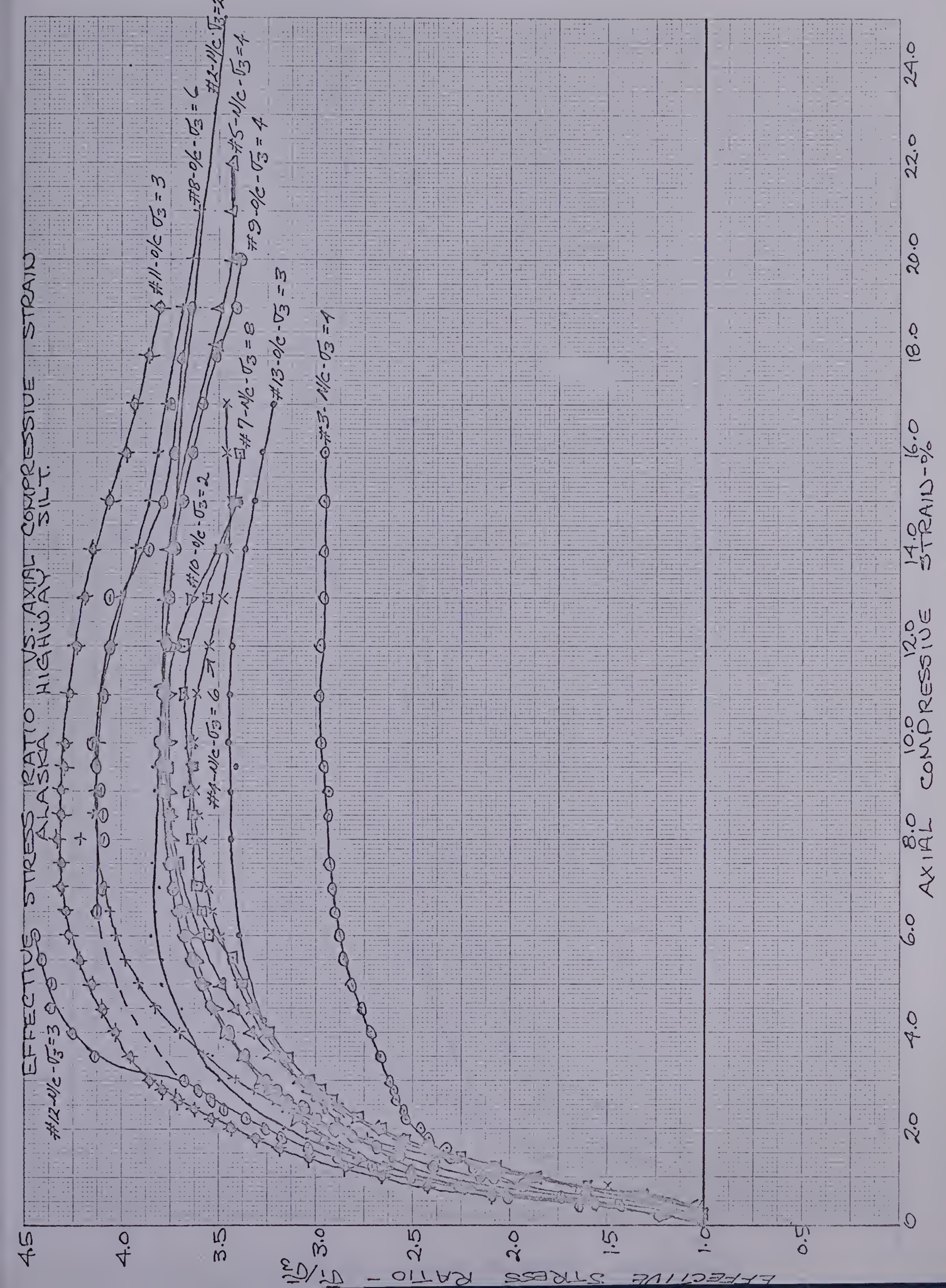
TEST RESULTS OBTAINED FOR
ALASKA HIGHWAY SILT, KAOLIN
MODIFIED SILT, AND MONTMORILLONITE
MODIFIED SILT



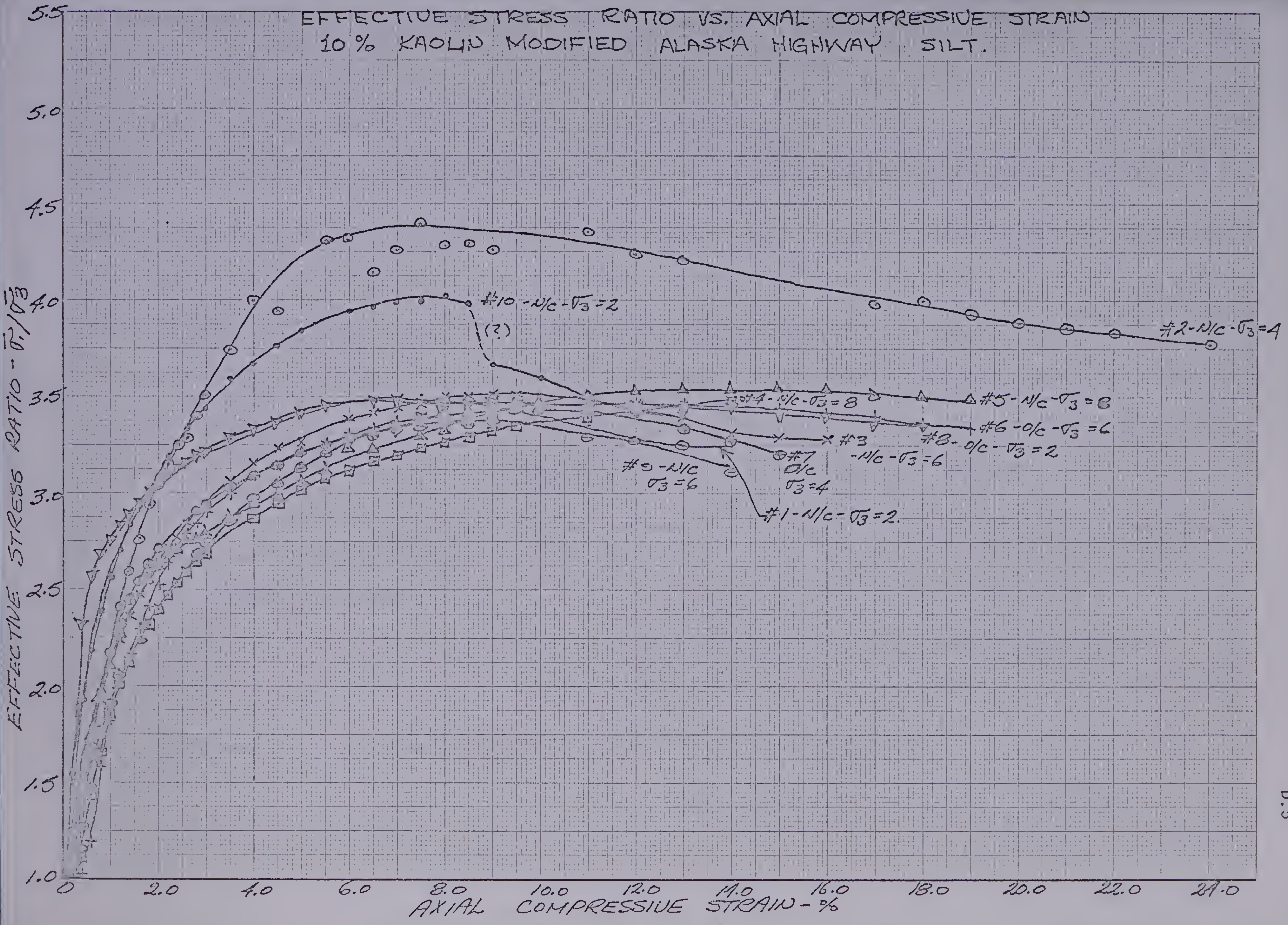
DEVIATOR STRESS VS. AXIAL COMPRESSIVE STRAIN
ALASKA HIGHWAY SILT
DEVIATOR STRESS-($\sigma_1-\sigma_3$)-Kg/cm^2.
AXIAL COMPRESSIVE STRAIN-%
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
#8-O/C-σ_3=6
#7-N/C-σ_3=8
#9-O/C-σ_3=4
#4-N/C-σ_3=6
#11-O/C-σ_3=3
#5-N/C-σ_3=4
#13-O/C-σ_3=3
#10-O/C-σ_3=2
#3-N/C-σ_3=4
#2
σ_3=2-N/C.
#12-N/C-σ_3=3

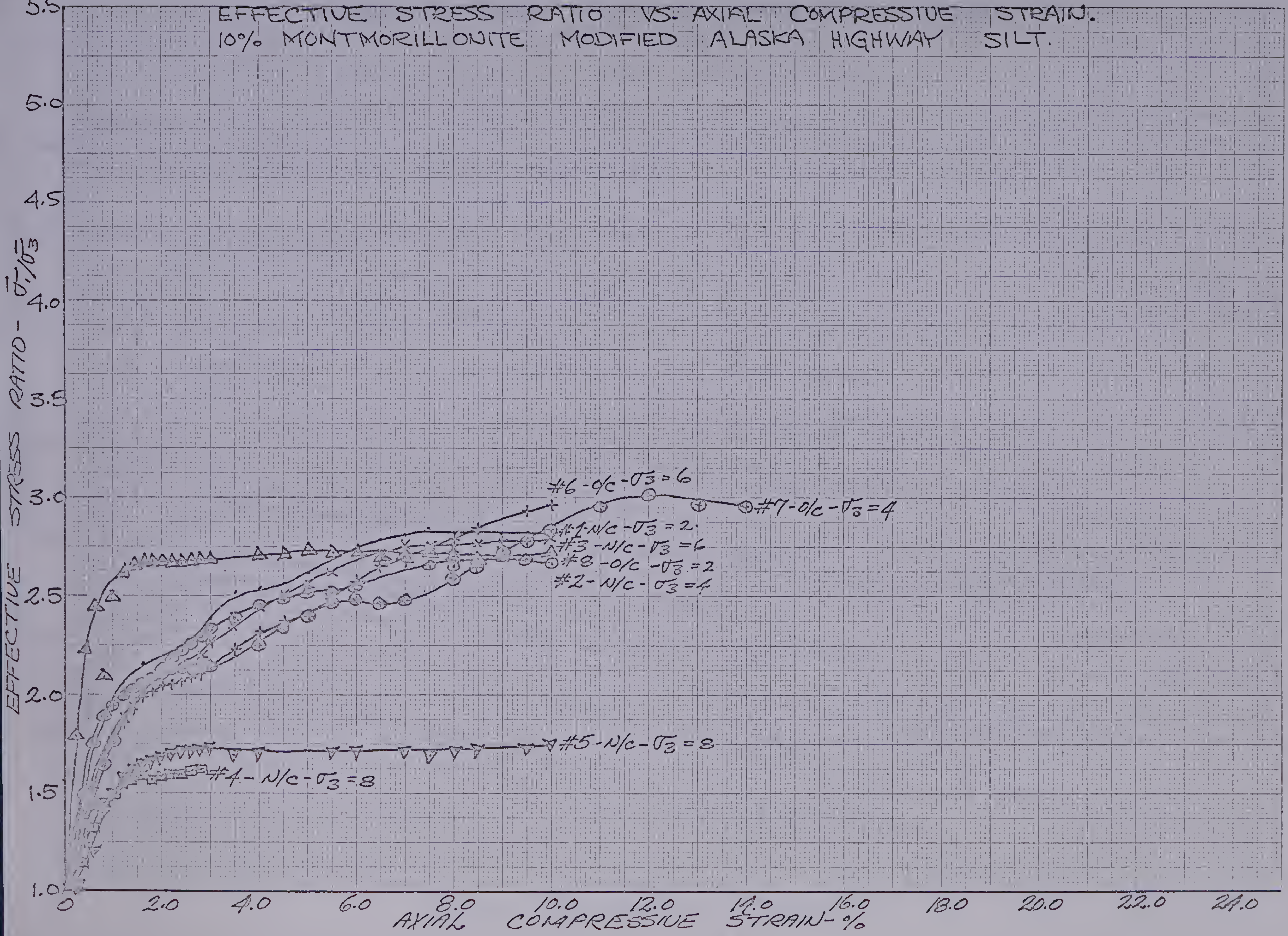

DEVIATOR STRESS VS. AXIAL COMPRESSIVE STRAIN
10% KAOLIN MODIFIED ALASKA HIGHWAY SILT
#4-N/C-σ3=8
#5-N/C-σ3=8
#6-O/C-σ3=6
#7-O/C-σ3=4
#3-N/C-σ3=6
#9-N/C-σ3=6
#8-O/C-σ3=2.
#2-N/C σ3=4
#1-N/C-σ3=2
#10-N/C-σ3=2.
AXIAL COMPRESSIVE STRAIN - %
DEVIATOR STRESS - (σ1-σ3) - KG/CM.²
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0

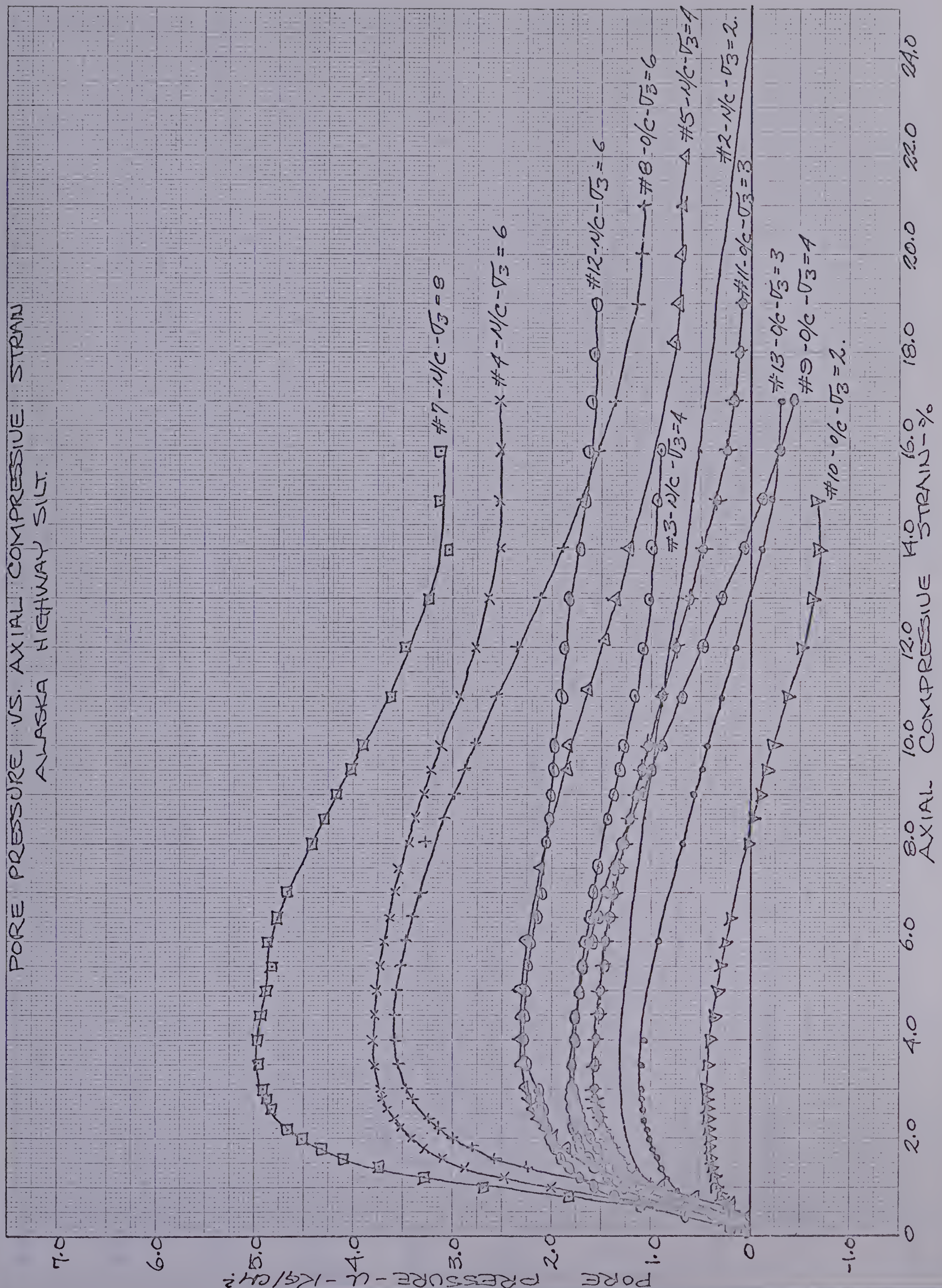

DEVIATOR STRESS VS. AXIAL COMPRESSIVE STRAIN
10% MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT.
DEVIATOR STRESS - (σ_1-σ_3) - KG/CM.²
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
AXIAL COMPRESSIVE STRAIN - %
#4 - N/C - σ_3 = 8
#5 - N/C - σ_3 = 8
#6 - O/C - σ_3 = 6
#7 - O/C - σ_3 = 4
#3 - N/C - σ_3 = 6
#8 - O/C - σ_3 = 2
#2 - N/C - σ_3 = 4
#1 - N/C - σ_3 = 2.

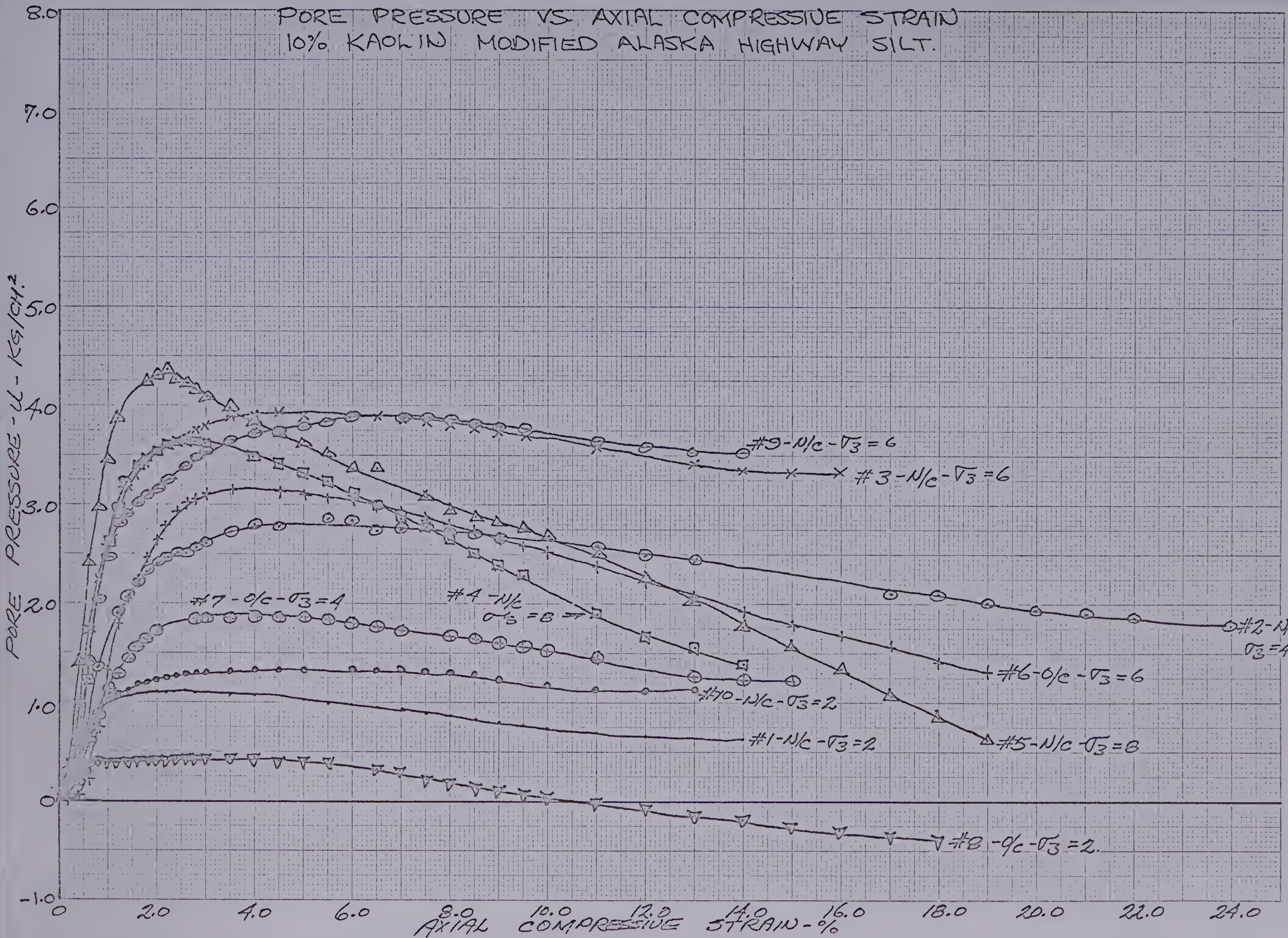


EFFECTIVE STRESS RATIO VS. AXIAL COMPRESSIVE STRAIN
ALASKA HIGHWAY SILT
#12-N/C-σ3=3
#11-O/C σ3=3
#8-O/C-σ3=6
#2-N/C-σ3=2
#5-N/C-σ3=4
#9-O/C-σ3=4
#7-N/C-σ3=8
#13-O/C-σ3=3
#10-O/C-σ3=2
#4-N/C-σ3=6
#3-N/C-σ3=4
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
AXIAL COMPRESSIVE STRAIN-%
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
EFFECTIVE STRESS RATIO - σ1/σ3

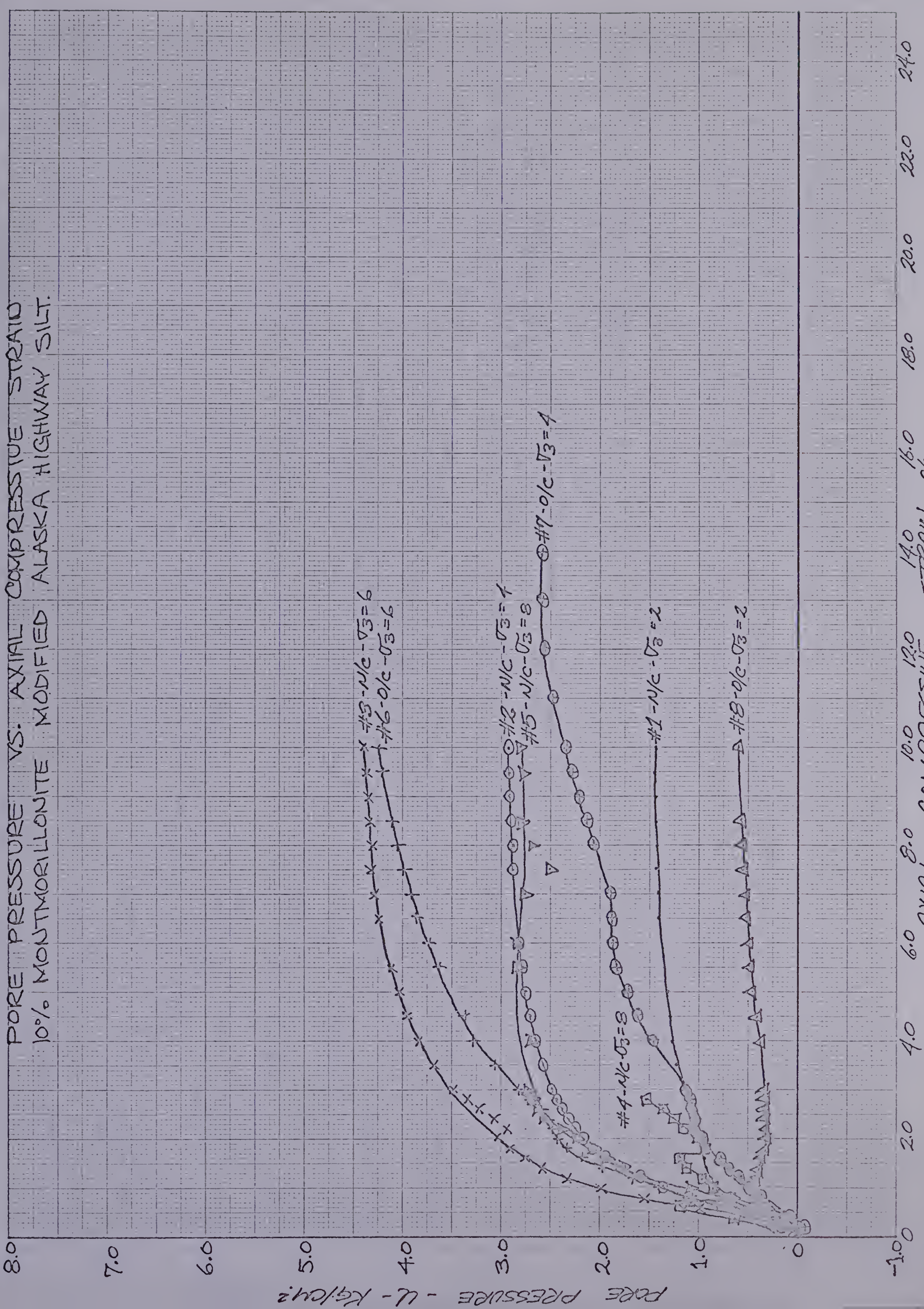

EFFECTIVE STRESS RATIO VS. AXIAL COMPRESSIVE STRAIN
10 % KAOLIN MODIFIED ALASKA HIGHWAY SILT.
EFFECTIVE STRESS RATIO - $\bar{\sigma}_1/\bar{\sigma}_3$
AXIAL COMPRESSIVE STRAIN - %
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
#10 - N/c - σ_3 = 2
(?)
#2 - N/c - σ_3 = 4
#5 - N/c - σ_3 = 8
#4 - N/c - σ_3 = 8
#6 - O/c - σ_3 = 6
#3 - N/c - σ_3 = 6
#8 - O/c - σ_3 = 2
#7 O/c σ_3 = 4
#9 - N/c σ_3 = 6
#1 - N/c - σ_3 = 2.


EFFECTIVE STRESS RATIO VS. AXIAL COMPRESSIVE STRAIN.
10% MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT.
EFFECTIVE STRESS RATIO - $\bar{\sigma}_1/\bar{\sigma}_3$
AXIAL COMPRESSIVE STRAIN - %
#6 - O/C - σ_3 = 6
#7 - O/C - σ_3 = 4
#1 - N/C - σ_3 = 2
#3 - N/C - σ_3 = 6
#8 - O/C - σ_3 = 2
#2 - N/C - σ_3 = 4
#5 - N/C - σ_3 = 8
#4 - N/C - σ_3 = 8
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0

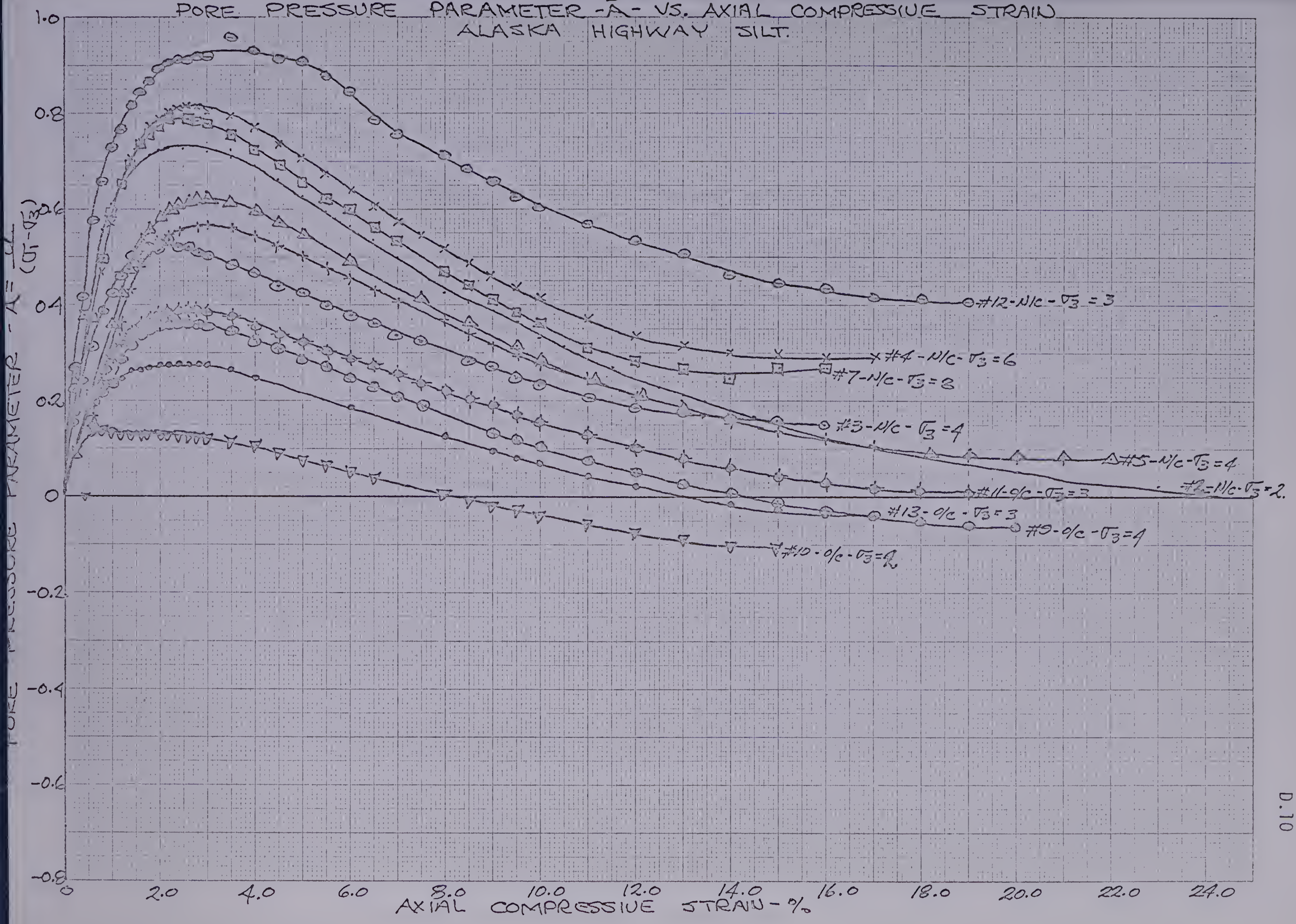

PORE PRESSURE VS. AXIAL COMPRESSIVE STRAIN
ALASKA HIGHWAY SILT
PORE PRESSURE - u - kg/cm²
AXIAL COMPRESSIVE STRAIN - %
#7 - N/C - σ3 = 8
#4 - N/C - σ3 = 6
#12 - N/C - σ3 = 6
#8 - O/C - σ3 = 6
#5 - N/C - σ3 = 4
#2 - N/C - σ3 = 2.
#3 - N/C - σ3 = 4
#11 - O/C - σ3 = 3
#13 - O/C - σ3 = 3
#9 - O/C - σ3 = 4
#10 - O/C - σ3 = 2.
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
-1.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0

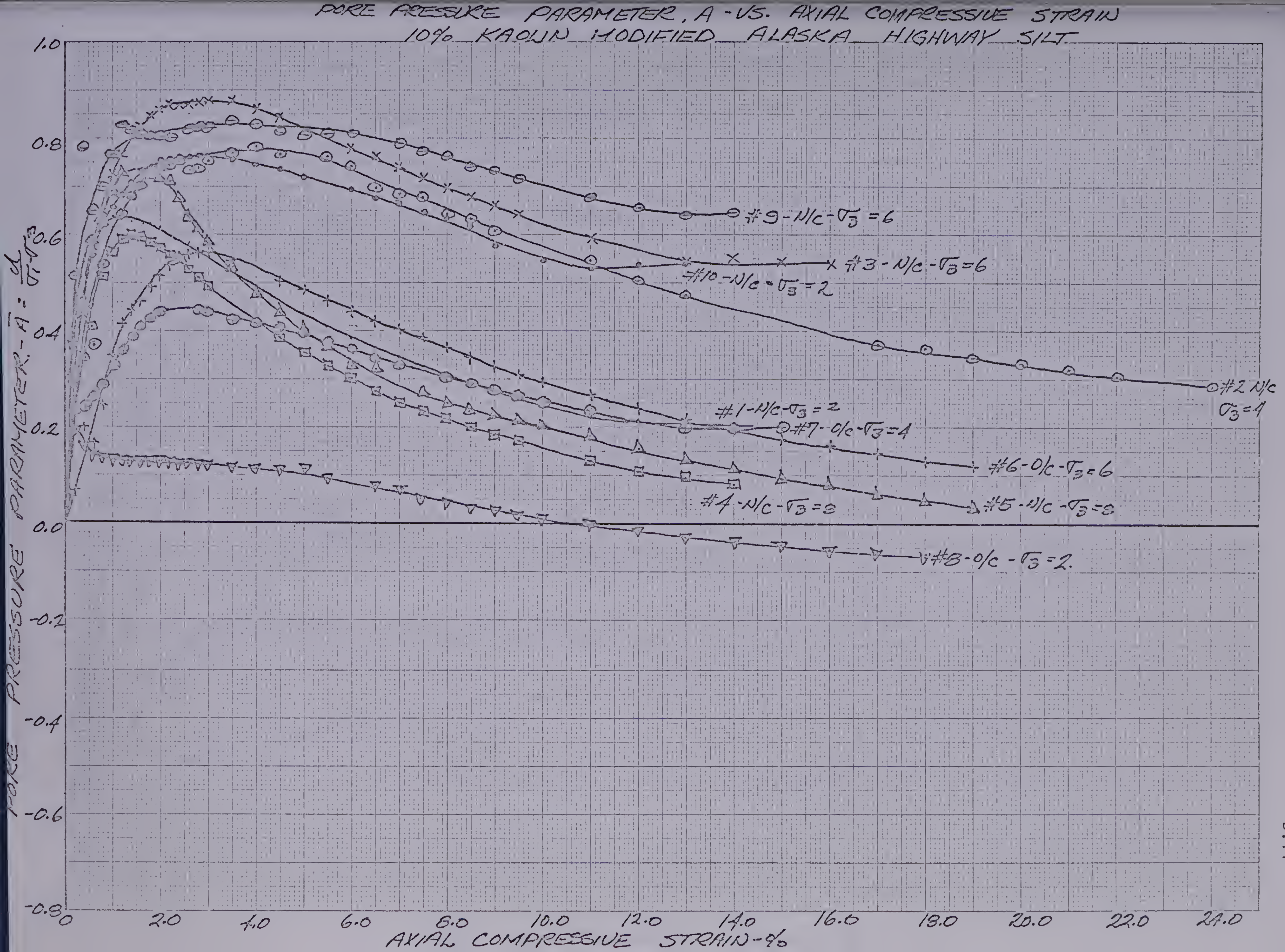


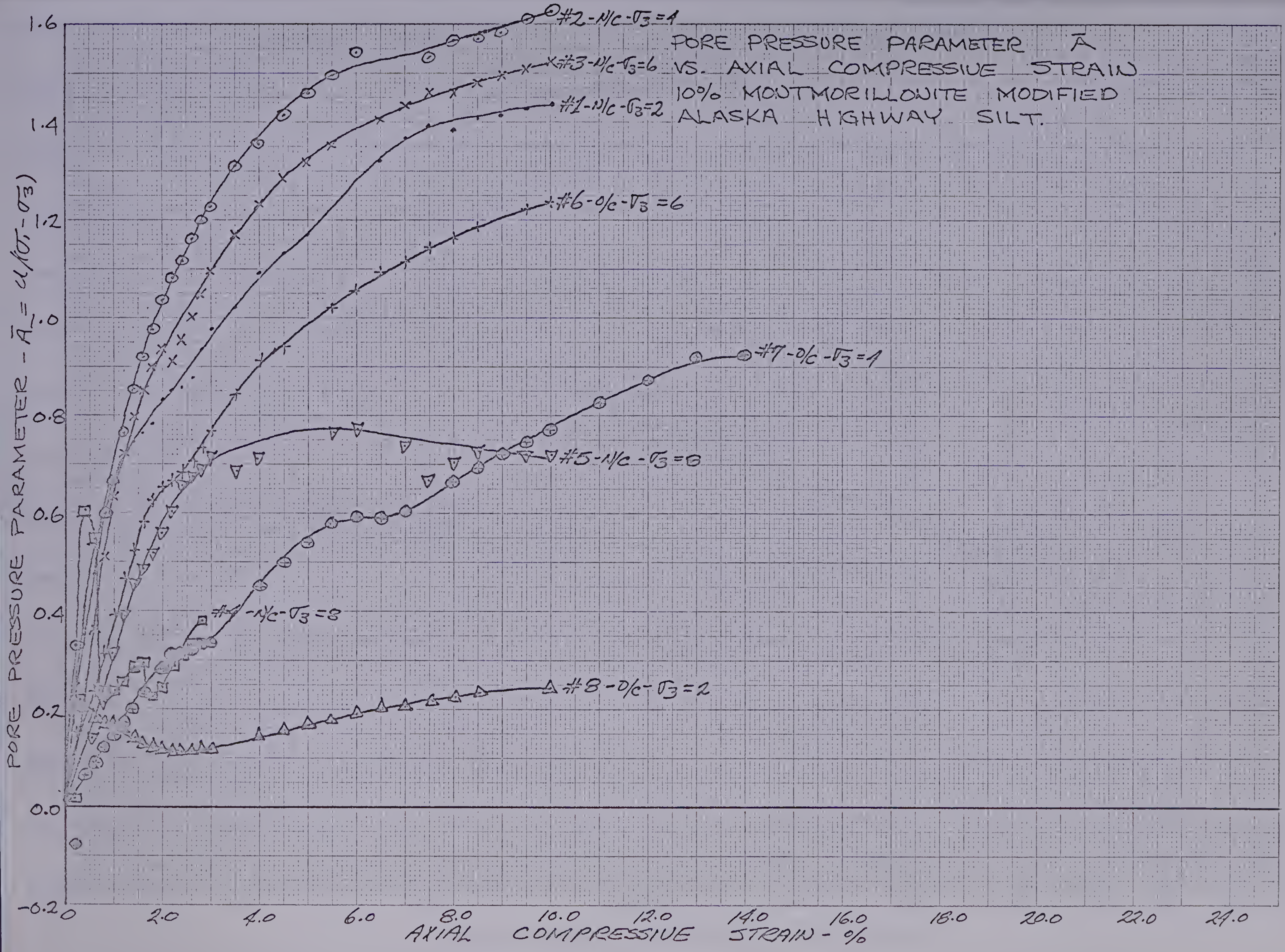
PORE PRESSURE VS. AXIAL COMPRESSIVE STRAIN
10% KAOLIN MODIFIED ALASKA HIGHWAY SILT.
PORE PRESSURE - u - KG/CM.²
AXIAL COMPRESSIVE STRAIN - %
#9 - N/c - σ3 = 6
#3 - N/c - σ3 = 6
#7 - O/c - σ3 = 4
#4 - N/c σ3 = 8
#2 - N/c σ3 = 4
#6 - O/c - σ3 = 6
#10 - N/c - σ3 = 2
#1 - N/c - σ3 = 2
#5 - N/c - σ3 = 8
#8 - O/c - σ3 = 2.
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
-1.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0

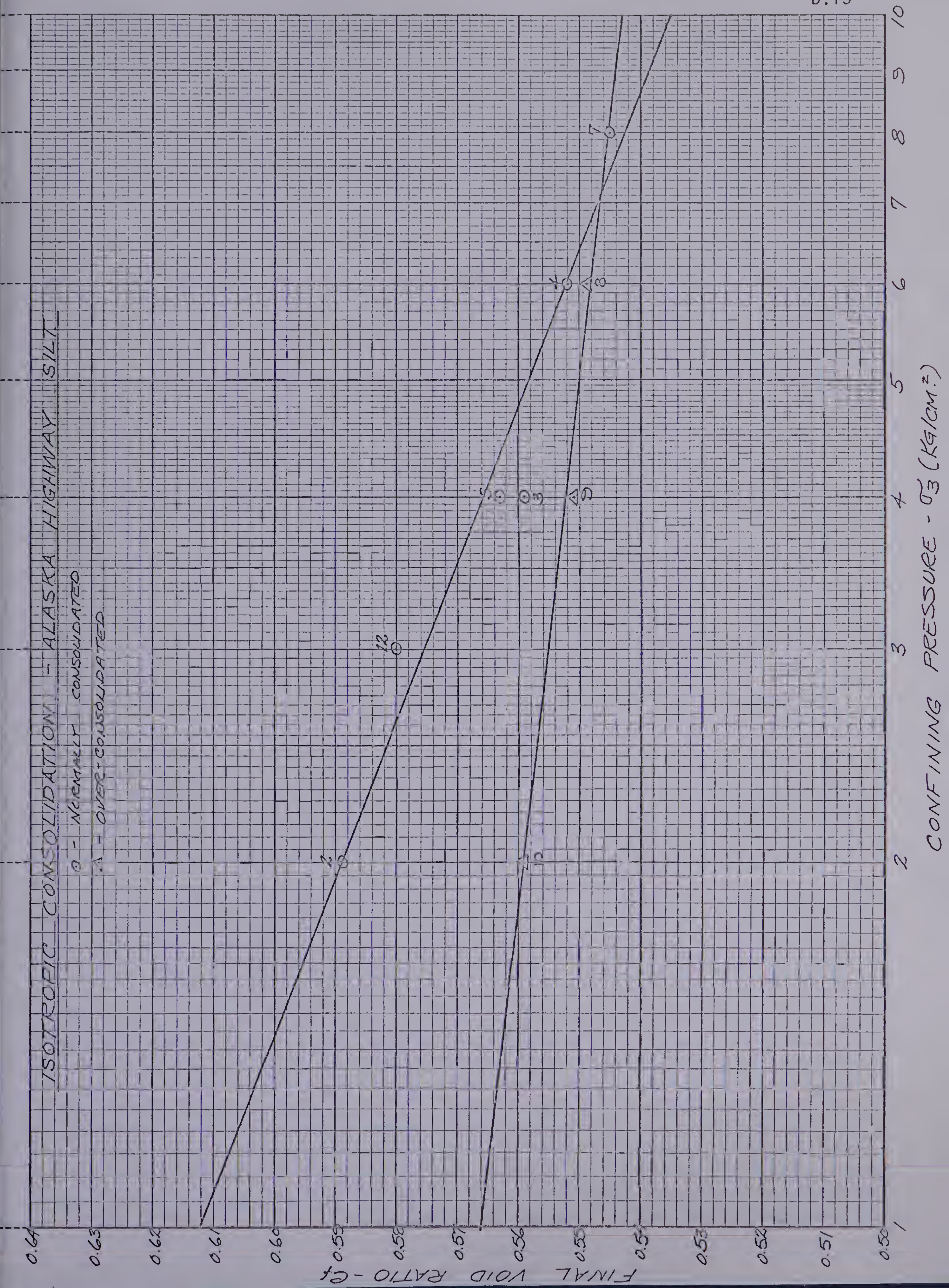


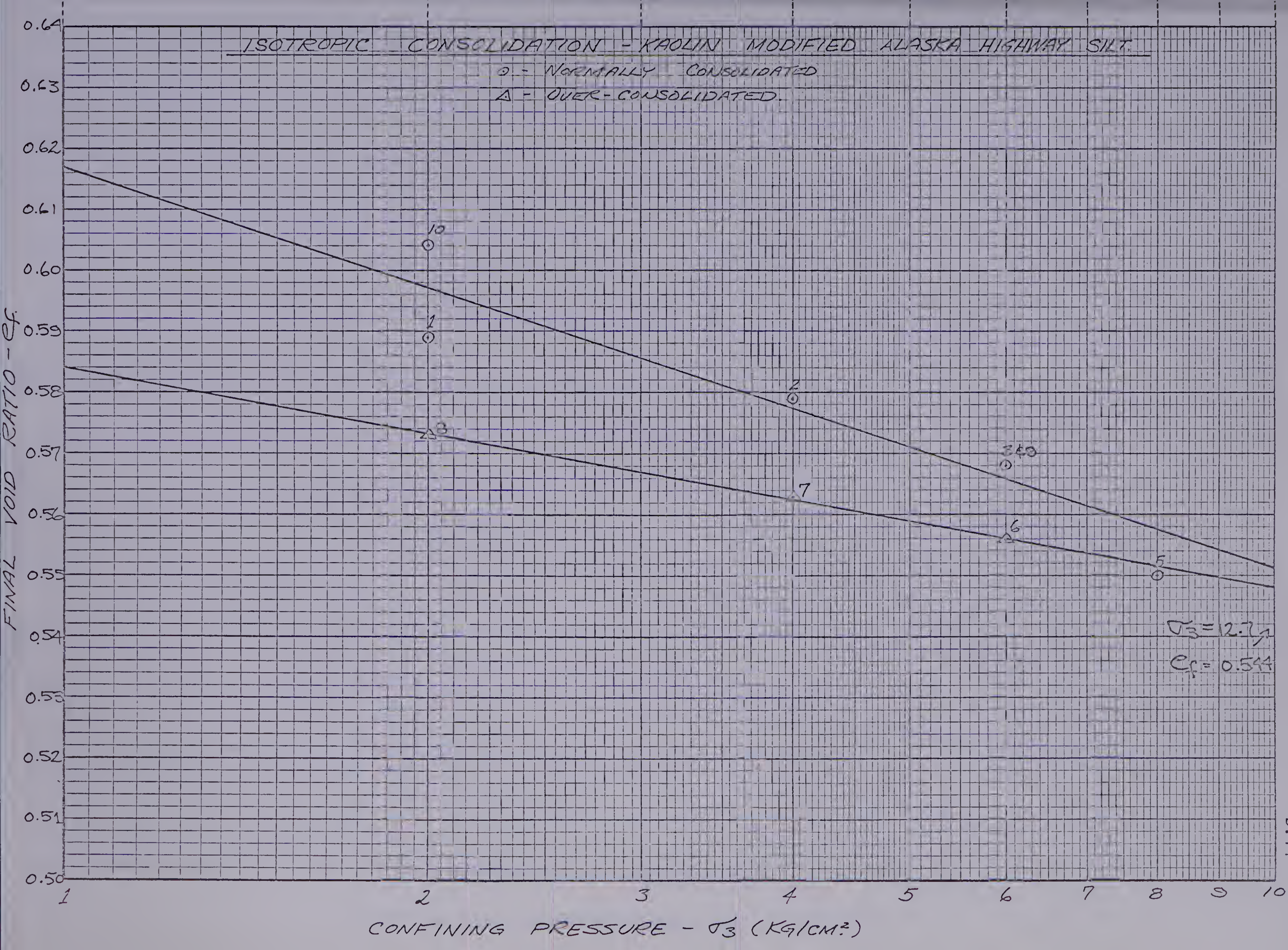
PORE PRESSURE VS. AXIAL COMPRESSIVE STRAIN
10% MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT.
#3-N/C-σ_3=6
#6-O/C-σ_3=6
#2-N/C-σ_3=4
#5-N/C-σ_3=8
#7-O/C-σ_3=4
#4-N/C-σ_3=8
#1-N/C-σ_3=2
#8-O/C-σ_3=2
AXIAL COMPRESSIVE STRAIN - %
PORE PRESSURE - U - Kg/cm²
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
-1.0
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0

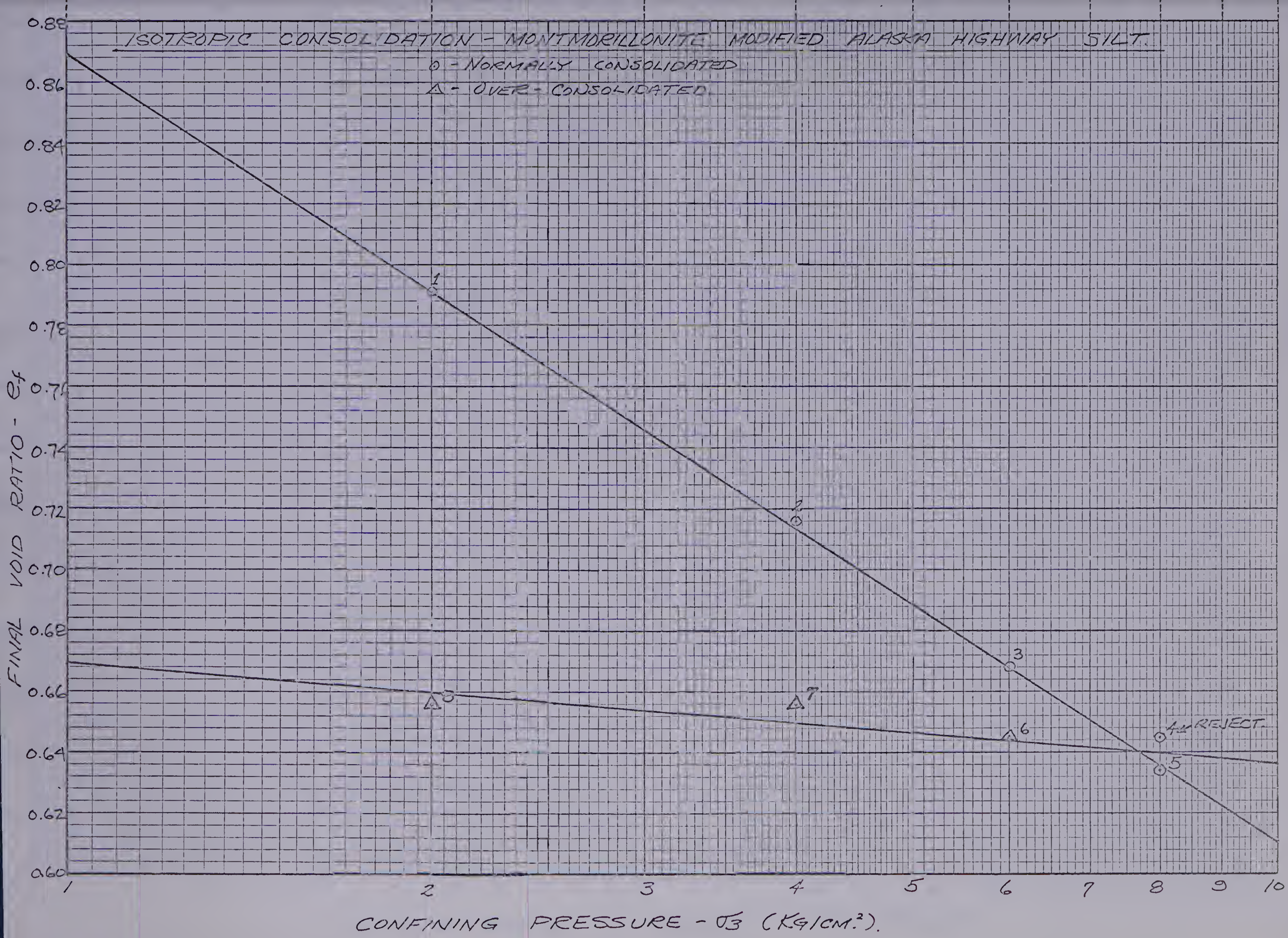

PORE PRESSURE PARAMETER -Ā- VS. AXIAL COMPRESSIVE STRAIN
ALASKA HIGHWAY SILT.
PORE PRESSURE PARAMETER - Ā = u/(σ1-σ3)
AXIAL COMPRESSIVE STRAIN - %
#12-N/c-σ3=3
#4-N/c-σ3=6
#7-N/c-σ3=8
#3-N/c-σ3=4
#5-N/c-σ3=4
#2-N/c-σ3=2.
#11-O/c-σ3=3
#13-O/c-σ3=3
#9-O/c-σ3=4
#10-O/c-σ3=2
1.0
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0

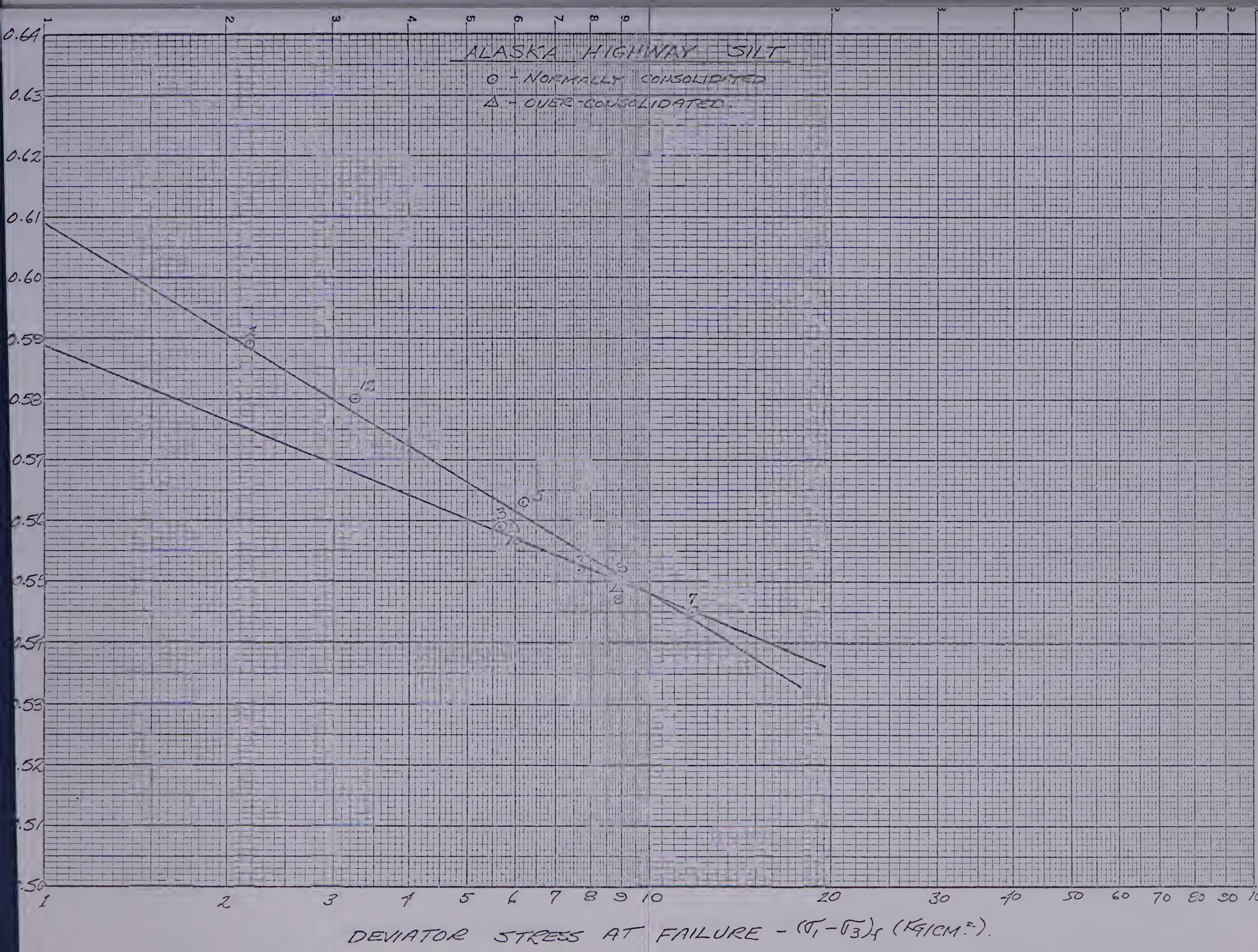

PORE PRESSURE PARAMETER, A -VS. AXIAL COMPRESSIVE STRAIN
10% KAOLIN MODIFIED ALASKA HIGHWAY SILT
PORE PRESSURE PARAMETER - $\bar{A} = \frac{u}{\sigma_1 - \sigma_3}$
AXIAL COMPRESSIVE STRAIN - %
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
#9 - N/c - σ_3 = 6
#3 - N/c - σ_3 = 6
#10 - N/c - σ_3 = 2
#2 N/c σ_3 = 4
#1 - N/c - σ_3 = 2
#7 - O/c - σ_3 = 4
#6 - O/c - σ_3 = 6
#4 - N/c - σ_3 = 8
#5 - N/c - σ_3 = 8
#8 - O/c - σ_3 = 2.

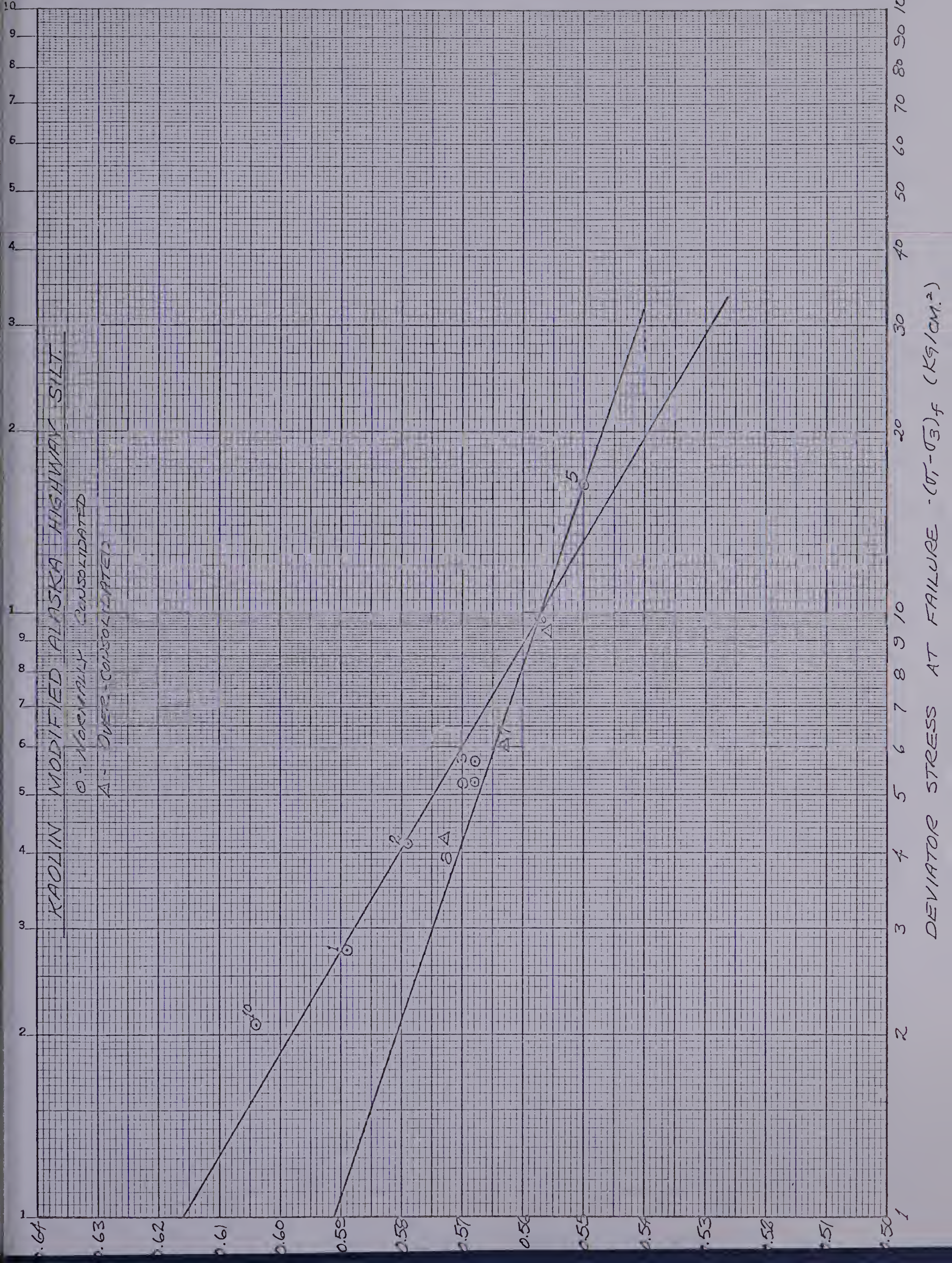

PORE PRESSURE PARAMETER $\bar{A}$
VS. AXIAL COMPRESSIVE STRAIN
10% MONTMORILLONITE MODIFIED
ALASKA HIGHWAY SILT.
#2-N/c-σ_3=1
#3-N/c-σ_3=6
#1-N/c-σ_3=2
#6-O/c-σ_3=6
#7-O/c-σ_3=1
#5-N/c-σ_3=8
#4-N/c-σ_3=8
#8-O/c-σ_3=2
PORE PRESSURE PARAMETER - $\bar{A}_1 = u/(\sigma_1 - \sigma_3)$
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
AXIAL COMPRESSIVE STRAIN - %

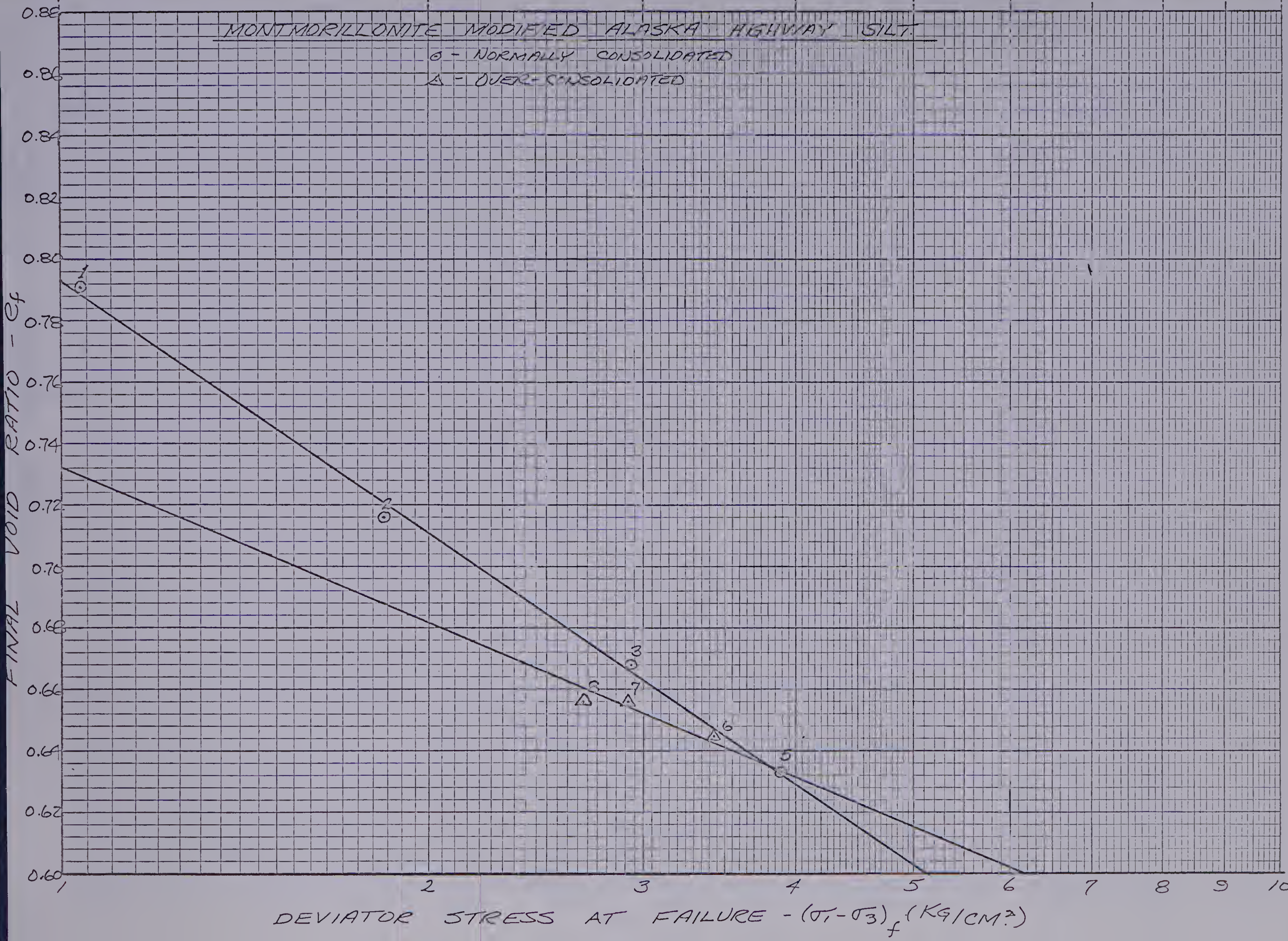

ISOTROPIC CONSOLIDATION - ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED
△ - OVER-CONSOLIDATED
CONFINING PRESSURE - σ_3 (KG/CM.²)
FINAL VOID RATIO - e_f
1
2
3
4
5
6
7
8
9
10
0.64
0.63
0.62
0.61
0.60
0.59
0.58
0.57
0.56
0.55
0.54
0.53
0.52
0.51
0.50


ISOTROPIC CONSOLIDATION - KAOLIN MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED
Δ - OVER-CONSOLIDATED.
FINAL VOID RATIO - e_f
CONFINING PRESSURE - σ_3 (KG/CM²)
$\sigma_3 = 12.7$
$e_f = 0.544$
0.64
0.63
0.62
0.61
0.60
0.59
0.58
0.57
0.56
0.55
0.54
0.53
0.52
0.51
0.50
1
2
3
4
5
6
7
8
9
10
10
1
2
3 & 9
8
7
6
5

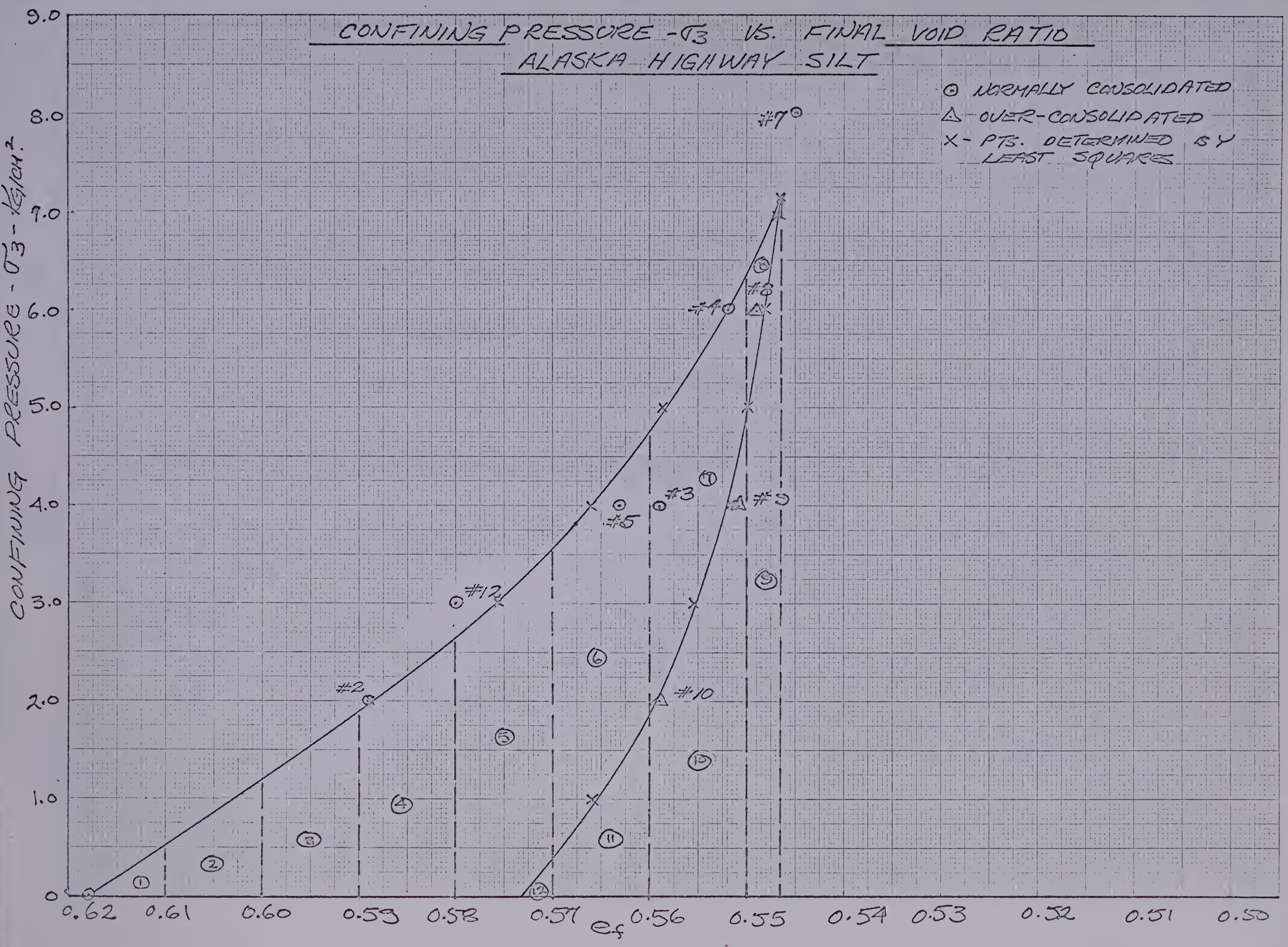

ISOTROPIC CONSOLIDATION - MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT.
⊙ - NORMALLY CONSOLIDATED
△ - OVER - CONSOLIDATED.
1
2
3
4 REJECT.
5
6
7
8
0.88
0.86
0.84
0.82
0.80
0.78
0.76
0.74
0.72
0.70
0.68
0.66
0.64
0.62
0.60
1
2
3
4
5
6
7
8
9
10
FINAL VOID RATIO - e_f
CONFINING PRESSURE - σ_3 (KG/CM.²).

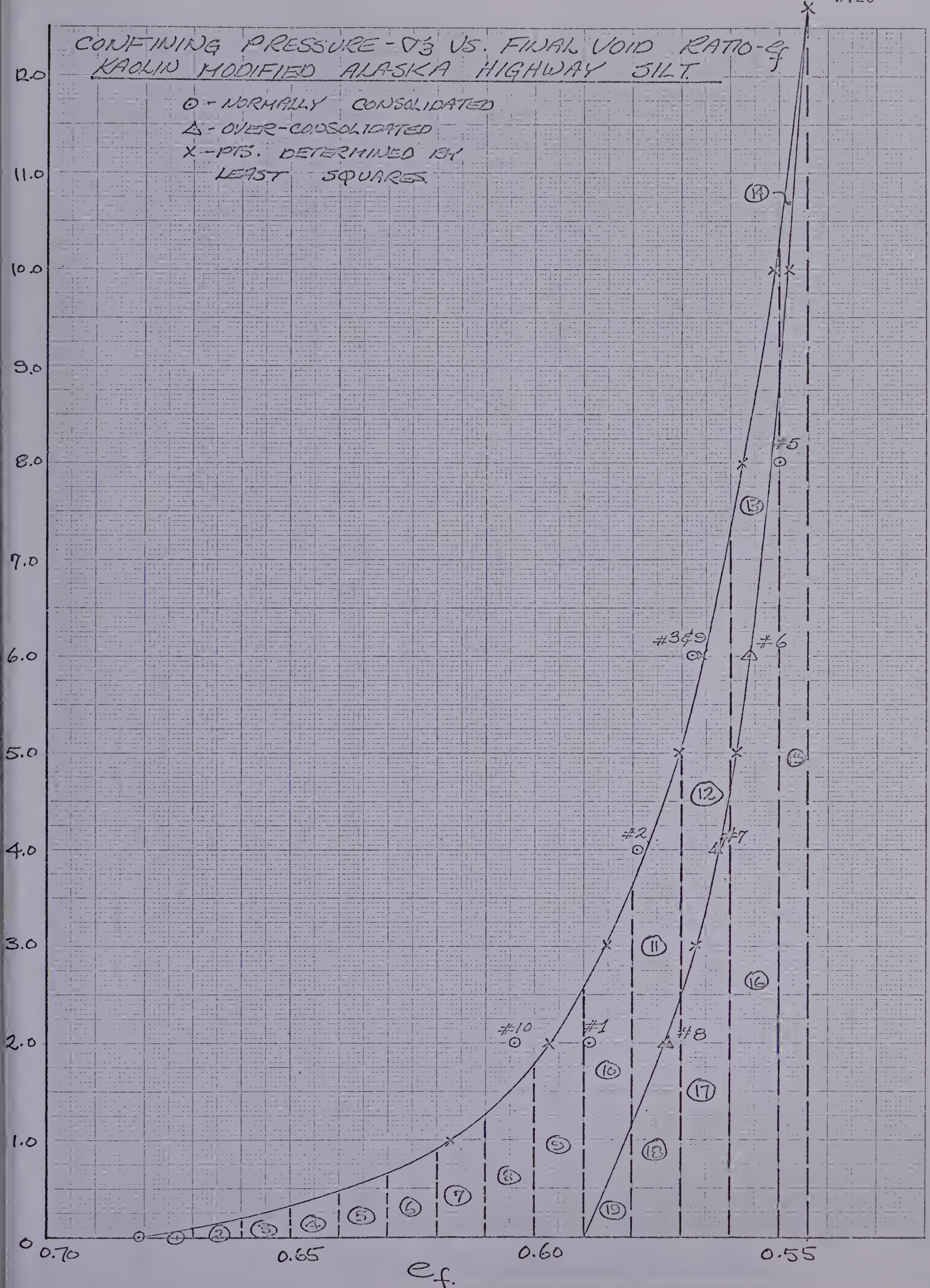

ALASKA HIGHWAY SILT
○ - NORMALLY CONSOLIDATED
△ - OVER-CONSOLIDATED.
0.64
0.63
0.62
0.61
0.60
0.59
0.58
0.57
0.56
0.55
0.54
0.53
0.52
0.51
0.50
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
DEVIATOR STRESS AT FAILURE - $(\sigma_1-\sigma_3)_f$ (KG/CM²).

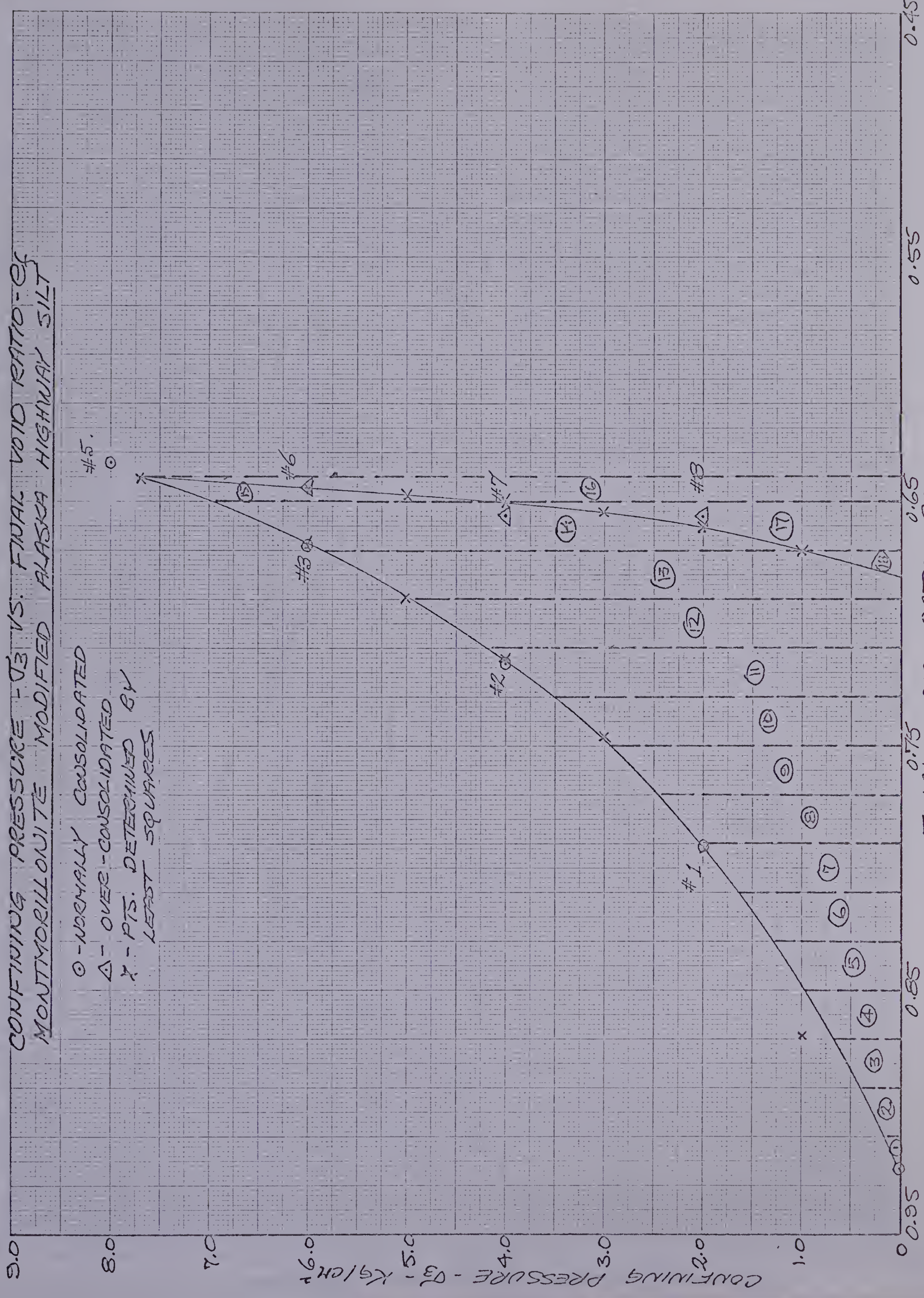

KAOLIN MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED
△ - OVER-CONSOLIDATED
DEVIATOR STRESS AT FAILURE - $(\sigma_1-\sigma_3)_f$ (Kg/CM.²)
0.64
0.63
0.62
0.61
0.60
0.59
0.58
0.57
0.56
0.55
0.54
0.53
0.52
0.51
0.50
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100

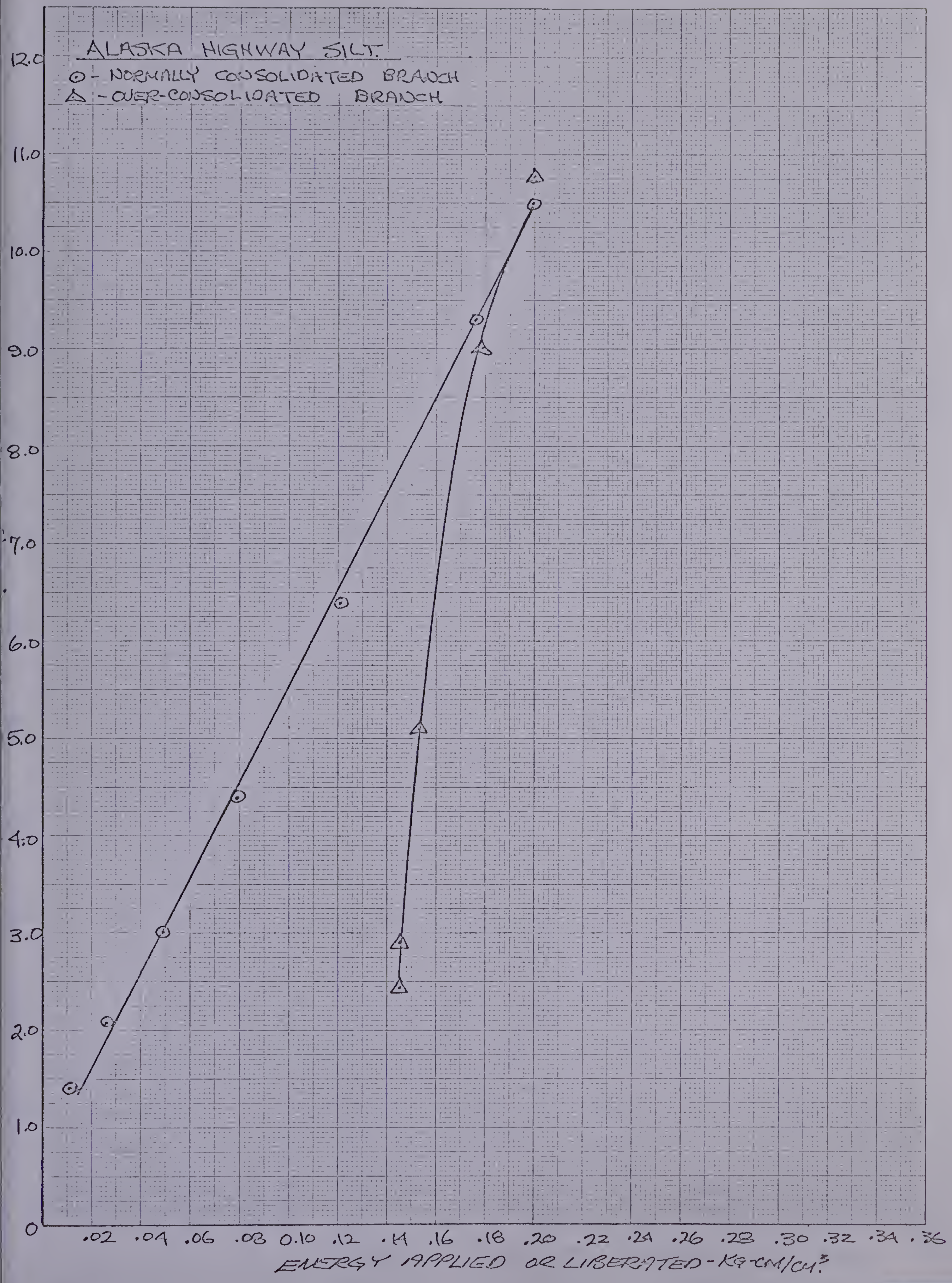


MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED
△ - OVER-CONSOLIDATED
FINAL VOID RATIO - e_f
DEVIATOR STRESS AT FAILURE - $(\sigma_1 - \sigma_3)_f$ (KG/CM²)
0.88
0.86
0.84
0.82
0.80
0.78
0.76
0.74
0.72
0.70
0.68
0.66
0.64
0.62
0.60
1
2
3
4
5
6
7
8
9
10

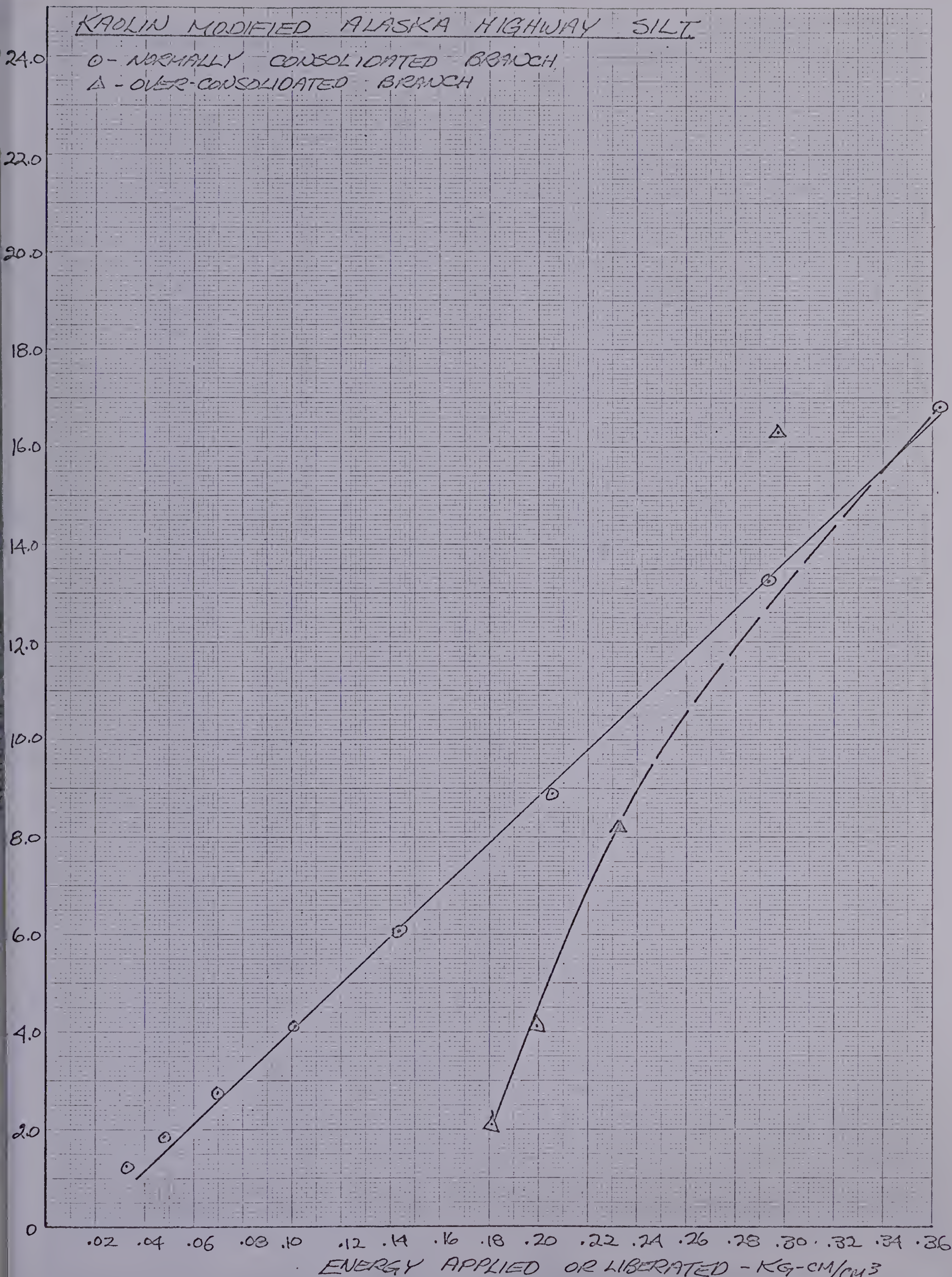

CONFINING PRESSURE - σ_3 VS. FINAL VOID RATIO
ALASKA HIGHWAY SILT
⊙ NORMALLY CONSOLIDATED
△ OVER-CONSOLIDATED
X - PTS. DETERMINED BY LEAST SQUARES
CONFINING PRESSURE - σ_3 - KG/CM.²
e_f
#7
#8
#4
#9
#3
#5
#12
#2
#10
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
0.62
0.61
0.60
0.59
0.58
0.57
0.56
0.55
0.54
0.53
0.52
0.51
0.50

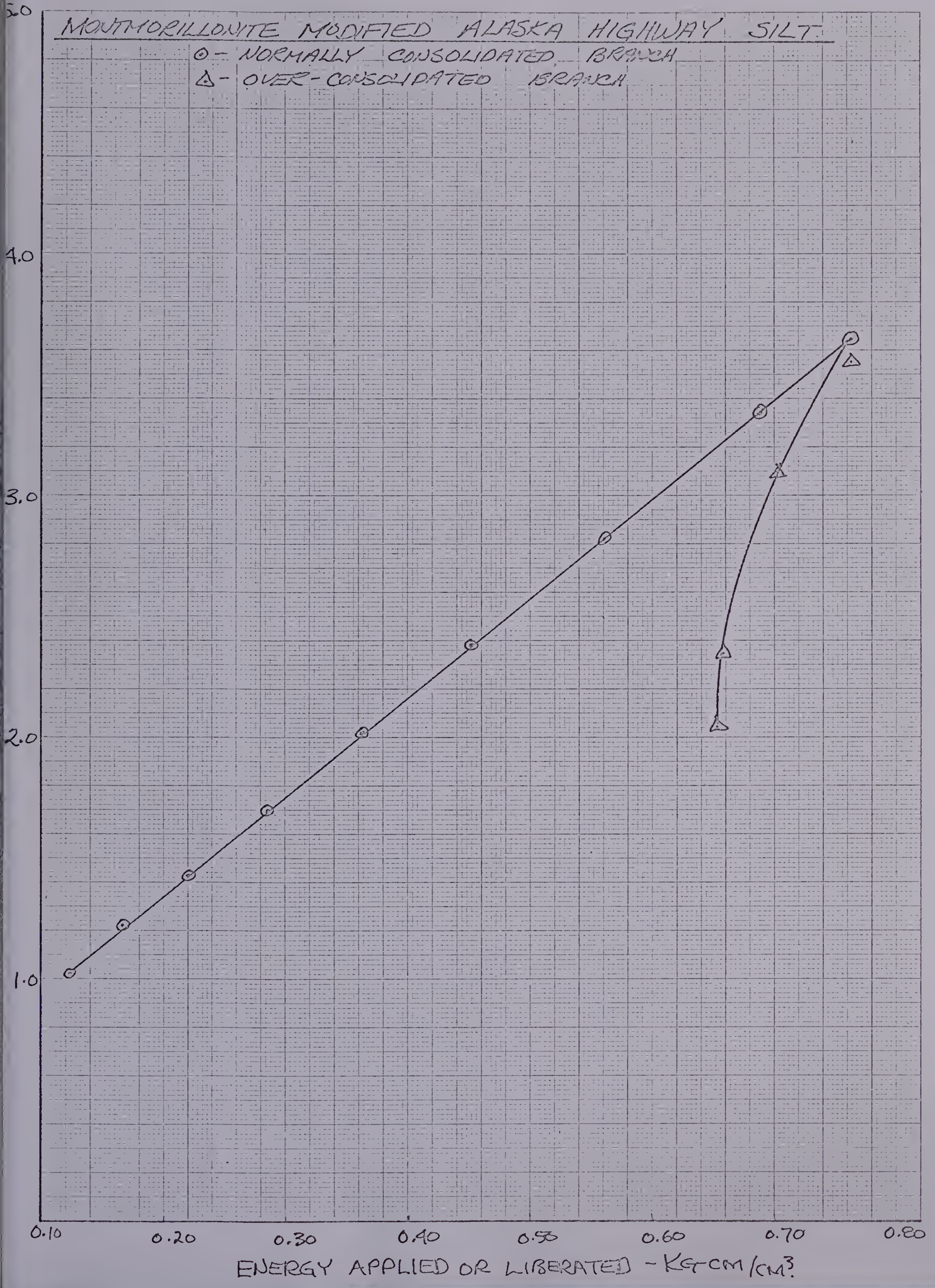

CONFINING PRESSURE - σ3 VS. FINAL VOID RATIO - ef
KAOLIN MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED
△ - OVER-CONSOLIDATED
X - PTS. DETERMINED BY LEAST SQUARES
#1
#2
#3 & 9
#5
#6
#7
#8
#10
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
0.70
0.65
0.60
0.55
ef


CONFINING PRESSURE - σ3 VS. FINAL VOID RATIO - ef
MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED
△ - OVER-CONSOLIDATED
X - PTS. DETERMINED BY LEAST SQUARES
CONFINING PRESSURE - σ3 - KG/CM²
FINAL VOID RATIO - ef
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
0.95
0.85
0.75
0.65
0.55
0.45
#1
#2
#3
#5.
#6
#7
#8



ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED BRANCH
△ - OVER-CONSOLIDATED BRANCH
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
.02 .04 .06 .08 0.10 .12 .14 .16 .18 .20 .22 .24 .26 .28 .30 .32 .34 .36
ENERGY APPLIED OR LIBERATED - KG-CM/CM.³


KAOLIN MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED BRANCH
△ - OVER-CONSOLIDATED BRANCH
24.0
22.0
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
.02 .04 .06 .08 .10 .12 .14 .16 .18 .20 .22 .24 .26 .28 .30 .32 .34 .36
ENERGY APPLIED OR LIBERATED - KG-CM/CM³


MONTMORILLONITE MODIFIED ALASKA HIGHWAY SILT
⊙ - NORMALLY CONSOLIDATED BRANCH
△ - OVER-CONSOLIDATED BRANCH
5.0
4.0
3.0
2.0
1.0
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
ENERGY APPLIED OR LIBERATED - KG-CM/CM³